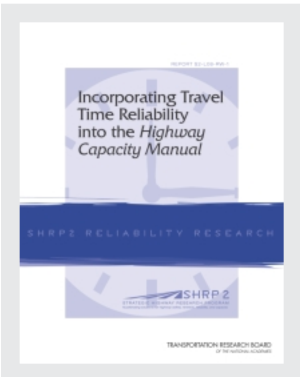

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SHRP 2 REPORT S2-L08-RW-1

Incorporating Travel Time Reliability into the *Highway Capacity Manual*

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Highways
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The Second Strategic Highway Research Program

America's highway system is critical to meeting the mobility and economic needs of local communities, regions, and the nation. Developments in research and technology—such as advanced materials, communications technology, new data collection technologies, and human factors science—offer a new opportunity to improve the safety and reliability of this important national resource. Breakthrough resolution of significant transportation problems, however, requires concentrated resources over a short time frame. Reflecting this need, the second Strategic Highway Research Program (SHRP 2) has an intense, large-scale focus, integrates multiple fields of research and technology, and is fundamentally different from the broad, mission-oriented, discipline-based research programs that have been the mainstay of the highway research industry for half a century.

The need for SHRP 2 was identified in *TRB Special Report 260: Strategic Highway Research: Saving Lives, Reducing Congestion, Improving Quality of Life,* published in 2001 and based on a study sponsored by Congress through the Transportation Equity Act for the 21st Century (TEA-21). SHRP 2, modeled after the first Strategic Highway Research Program, is a focused, time-constrained, management-driven program designed to complement existing highway research programs. SHRP 2 focuses on applied research in four areas: Safety, to prevent or reduce the severity of highway crashes by understanding driver behavior; Renewal, to address the aging infrastructure through rapid design and construction methods that cause minimal disruptions and produce lasting facilities; Reliability, to reduce congestion through incident reduction, management, response, and mitigation; and Capacity, to integrate mobility, economic, environmental, and community needs in the planning and designing of new transportation capacity.

SHRP 2 was authorized in August 2005 as part of the Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU). The program is managed by the Transportation Research Board (TRB) on behalf of the National Research Council (NRC). SHRP 2 is conducted under a memorandum of understanding among the American Association of State Highway and Transportation Officials (AASHTO), the Federal Highway Administration (FHWA), and the National Academy of Sciences, parent organization of TRB and NRC. The program provides for competitive, merit-based selection of research contractors; independent research project oversight; and dissemination of research results.

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Kittelson & Associates, Inc., was the primary contractor for the project and was supported by the following subcontractors: the Institute for Transportation Research and Education (ITRE) at North Carolina State University, Cambridge Systematics, Inc., the Texas A&M Research Foundation, and Write Rhetoric.

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FOREWORD

William Hyman, *SHRP 2 Senior Program Officer, Reliability*

The scope of work for SHRP 2 Reliability Project L08, Incorporating Travel Time Reliability into the *Highway Capacity Manual*, called for developing methods that could potentially address travel time reliability in the analytic procedures for freeway facilities and urban streets in the *Highway Capacity Manual* (HCM). This research resulted in two chapters that the TRB Committee on Capacity and Quality of Service has approved for inclusion in the HCM. The first is Chapter 36, concerning freeway facilities and urban streets, and the second is a supplemental Chapter 37, which provides more detail on the methodologies. Corresponding to the methodologies for incorporating reliability into the *Highway Capacity Manual* for freeway facilities and urban streets are new computational engines developed in conjunction with the 2010 HCM that work with FREEVAL and STREETVAL.

After this project was completed, the SHRP 2 Reliability Program conducted four pilots in the states of California, Florida, Minnesota, and Washington, in part to test the new computational software for addressing reliability. Recommendations for improving the software were compiled and selected enhancements were made, generally to make the software more user friendly.

While SHRP 2 finished the enhancements, further work on improving the reliability highway capacity analysis products has occurred under National Cooperative Highway Research Program (NCHRP) Project 03-115, Production of a Major Update to the *Highway Capacity Manual*. Those interested in the treatment of reliability in highway capacity analysis should examine this NCHRP work when it is completed. Interested individuals should also monitor the decisions of the TRB Committee on Highway Capacity and Quality of Service for approved updates to the manual.

The *Highway Capacity Manual* is one of the most widely consulted technical references in the transportation field. The work performed under SHRP 2 Reliability Project L08 is likely to allow decision makers, practitioners, and researchers to better understand the implications of nonrecurring congestion factors, such as incidents, weather, and work zones, for the capacity analysis, assessment of level of service, and performance evaluation of freeway facilities and urban streets.

CONTENTS

Executive Summary

This final report documents the activities performed during SHRP 2 Reliability Project L08: Incorporating Travel Time Reliability into the *Highway Capacity Manual.* It serves as a supplement to the proposed chapters for incorporating travel time reliability into the *Highway Capacity Manual* prepared for this same project. The proposed chapters demonstrate how to apply travel time reliability methods to the analysis of freeways, urban streets, and corridors. The final report summarizes the work activities conducted during the course of the Phase 1 and Phase 2 research by memorializing the activities, the processes, and the findings of the L08 project. In this way, the final report articulates the *how* and *why* of key decisions made and key activities undertaken during the project so that the logic and rationale are not lost to future researchers and practitioners who aim to build on the work completed in this effort.

This report addresses the following topics:

- The project's purpose, objectives, and work tasks (Chapter 1);
- The research team's proposed definition of *reliability,* along with means for measuring reliability (Chapter 2);
- A state-of-the-art and state-of-the-practice literature review (Chapter 3);
- An overview of the methodologies for calculating reliability for freeways and urban streets (Chapter 4);
- A description of the development of freeway and urban street scenario generators (Chapter 5);
- Enhancements to the *Highway Capacity Manual* (HCM) freeway facilities methodology and its computational engine FREEVAL-RL (FREeway EVALuation—ReLiability), and enhancements to the urban streets methodology and its computational engine STREETVAL (STREET eVALuation) (Chapter 6 and Appendices A and B);
- A procedure for conducting a corridor application (Chapter 7); and
- A method for future consideration to define levels of service using reliability as a service measure and a discussion of future research needs (Chapter 8).

The remainder of this executive summary provides a brief overview of the research results.

Definition for Travel Time Reliability

Travel time reliability aims to quantify the variation of travel time. It is defined using the entire range of travel times for a given trip, for a selected time period (e.g., the p.m. peak period on weekdays) over a selected horizon (e.g., a year). For the purpose of measuring reliability, a *trip* can be defined as occurring on a specific segment, facility (combination of multiple consecutive segments), or any subset of the transportation network; or the definition can be broadened to include a traveler's initial origin and final destination. Measuring travel time reliability requires

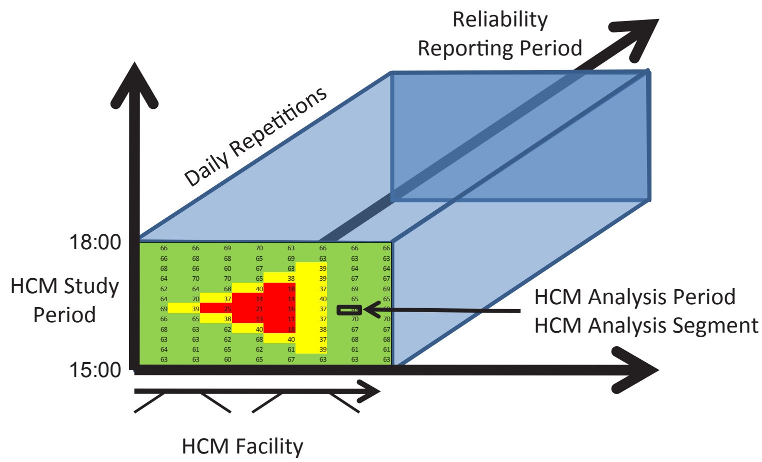


Figure ES.1. Reliability terms.

that a sufficient history of travel times be present to track travel time performance. This history is described by the travel time distribution for a given trip.

Once the travel time distribution is established, performance measures can be established to capture reliability. The two general types of reliability performance measures are the following:

1. Those that capture the variability in travel times that occurs for a trip over the course of time; and
2. Those that reflect the number of trips that fail or succeed according to a predetermined performance standard or schedule.

In both cases, reliability (more appropriately, unreliability) is caused by the interaction of the factors that influence travel times: fluctuations in demand (which may result from daily or seasonal variation, or special events), traffic control devices, traffic incidents, inclement weather, work zones, and physical capacity (based on prevailing geometrics and traffic patterns). These factors produce travel times that vary from day to day for the same trip.

The following terms, illustrated in Figure ES.1, are used throughout this report:

1. *Analysis period* is defined as the smallest time unit for which the HCM analysis procedure is applied. In the case of freeway and urban street facility analysis, the HCM analysis period is 15 min, although it can be of greater duration at the discretion of the analyst. Alternative tools may define different analysis period lengths.
2. *Study period* is defined as the sum of the sequential analysis periods for which the HCM facility analysis procedure is applied (e.g., a 4-hour peak period). The study period is defined by the analyst for each specific application, on the basis of the guidance provided in the HCM.
3. *Reliability reporting period* is defined as the period over which reliability is to be estimated (e.g., the 250 nonholiday weekdays in a year). In essence, the reliability reporting period specifies the number of days for which the reliability analysis is to be performed.

Reliability Metrics (for Use as Performance Measures)

Travel time reliability relates to how travel times for a given trip and time period perform over time. From a measurement perspective, reliability is quantified from the distribution of travel times—for a given facility or trip and the time period (e.g., weekday peak period)—which occurs over a significant span of time. One year is generally long enough to capture nearly all of the variability caused by disruptions. A variety of metrics can be computed once the travel time distribution has

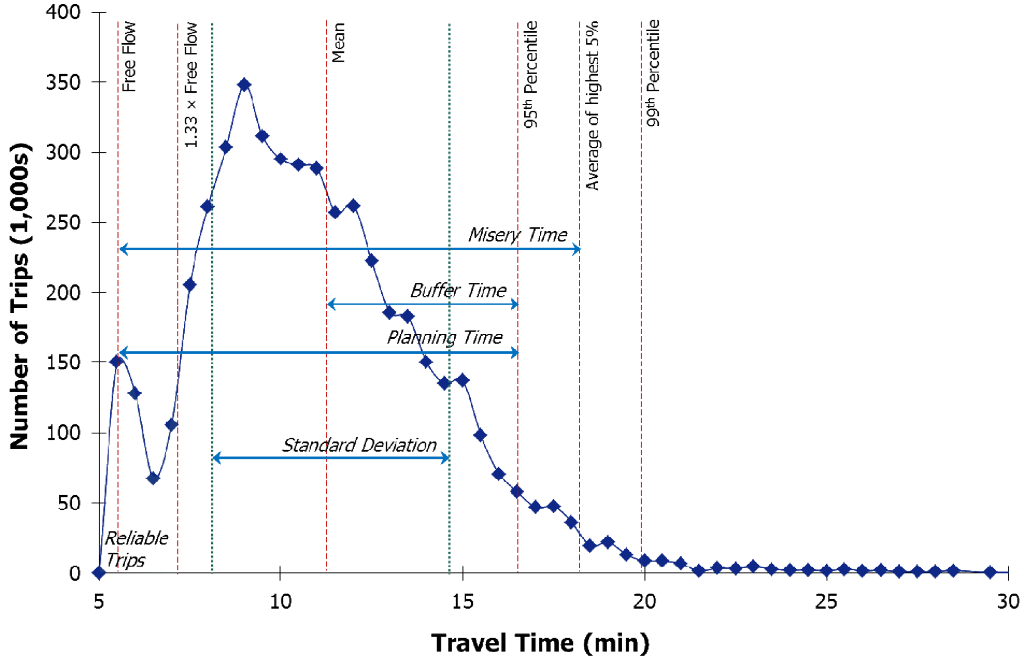


Figure ES.2. Travel time distribution as the basis for defining reliability metrics.

been established, including standard statistical measures (e.g., standard deviation, kurtosis), percentile-based measures (e.g., 95th percentile travel time, buffer index), on-time measures (e.g., percentage of trips completed within a travel time threshold), and failure measures (e.g., percentage of trips exceeding a travel time threshold). Some of these metrics are shown in Figure ES.2.

The set of performance measure metrics listed in Table ES.1 is recommended for Project L08. Both variability- and failure-based metrics are included. Which metric should be highlighted as the primary reliability metric is difficult to say. Much depends on the specific application being used. When interpreting Table ES.1, it should be noted that many of the selected performance measures are defined relative to the *free-flow* travel time, rather than the *average* travel time. This is deliberate because the average travel time (a) is not known before the analysis is conducted, (b) varies between different facilities, and (c) varies between different scenarios for the same

Table ES.1. Recommended Reliability Performance Measure Metrics for SHRP 2 Project L08

Reliability Performance Measure	Definition
Core Measure	
Reliability rating	Percentage of trips serviced at or below a threshold travel time index (TTI) (1.33 for freeways, 2.50 for urban streets)
Planning time index (PTI)	95th percentile TTI (95th percentile travel time divided by the free-flow travel time)
80th percentile TTI	80th percentile TTI (80th percentile travel time divided by the free-flow travel time)
Semistandard deviation	The standard deviation of travel time pegged to free-flow travel time rather than the mean travel time (variation is measured relative to free-flow travel time)
Failure or on-time measures	Percentage of trips with space mean speed less than 50, 45, and/or 30 mph
Supplemental Measure	
Standard deviation	Usual statistical definition
Misery index (modified)	The average of the highest 5% of travel times divided by the free-flow travel time

facility. Performance measures based on the average travel time are therefore deemed to be less appropriate for HCM analysis.

The distribution of travel times is the starting point for measuring reliability. In a statistical sense, the distribution is continuous only if it is based on measuring travel times from individual vehicles. As of this writing, the data used to monitor travel times—as well as modeling methods—rarely are managed in this way. For example, consider roadway detectors of spot speeds, which measure every vehicle that crosses their detection zone. These systems are designed to aggregate measurements in the field to 20- or 30-s summaries before transmission. So, in its lowest form, the speed "measurement" is really an average. The data are sometimes further aggregated to 1-, 5-, or 15-min summaries for archiving. At each aggregation, variability in the measurements is reduced. (When aggregating travel times over analysis periods, it is extremely important to weight the travel time averages by volume or vehicle miles traveled, rather than taking just the arithmetic mean.) Similarly, Bluetooth-based vehicle reidentification has a sampling rate well below 100%.

Methodology for Calculating Reliability

The objectives of SHRP 2 Project L08 are twofold. The first objective is to incorporate nonrecurring congestion effects into the HCM procedure. The second objective is to expand the analysis horizon from a single study period (typically an a.m. or p.m. peak period) to an extended time horizon of several weeks or months to assess the variability and the quality of service the facility provides to its users. This expanded period—referred to as the *reliability reporting period*—can be thought of as a series of consecutive days, each one having its own set of demands and capacities that affect the facility travel time. This study focused on weather, incidents, work zones, and special events on the supply side, and on volume variability by time of day, day of week, and month of year on the demand side.

Separate methodologies are used to evaluate reliability for freeway facilities and for urban streets, although many parallels exist between the two methods.

Freeway Facilities Methodology

At its highest level of representation, the freeway facilities methodology has three primary components: a data depository, a scenario generator, and a core computational procedure, which is an adapted and significantly revised version of the FREEVAL computational engine for reliability, or FREEVAL-RL. These components are illustrated in Figure ES.3.

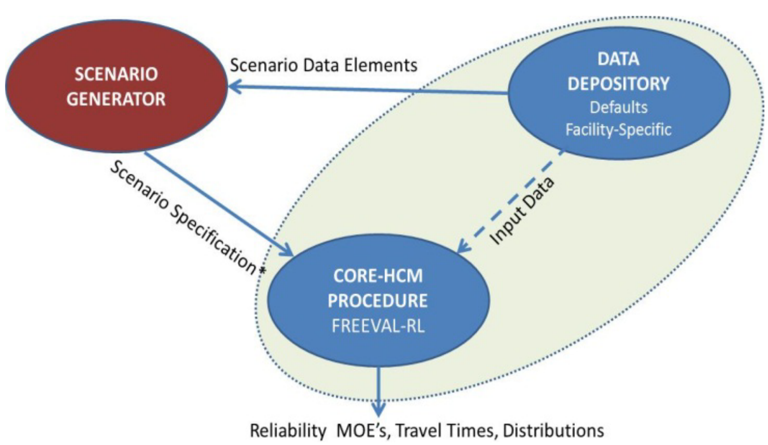


Figure ES.3. Freeway facilities methodology components, including measures of effectiveness (MOEs).

The largest shaded oval and dotted line represent the current implementation of the HCM freeway facilities chapter, with study period data specific to the facility being studied entered directly into FREEVAL-RL for analysis of (predominantly) recurring congestion effects. The connection to reliability is enabled by the addition of a scenario generator. Each component and its interaction with the other two are explained in some detail in the following sections.

The freeway scenario generator (FSG) developed by the L08 research team assigns initial probabilities to a number of base scenarios. A *base scenario probability* is expressed as the fraction of time a particular combination of events takes place during the study period (SP) of interest (e.g., a.m. or p.m. peak periods). In this project, a scenario is akin to a study period, which may or may not contain a given combination of weather or incident events. Base scenario probabilities are computed assuming independence between the events, and at that initial stage do not take into account the actual duration of the event (weather or incident) in question. They only take into account the categories of weather and/or incidents. (See Appendices C through G for information about recurring demand for the freeway scenario generator, weather and incident-related crash frequencies, and weather modeling.)

Urban Street Facilities Methodology

The reliability methodology for urban streets consists of the following three components:

- Scenario generation;
- Facility evaluation; and
- Performance summary.

These components are used in sequence to generate, evaluate, and summarize the various scenarios that make up the reliability reporting period. The HCM2010 urban streets methodology (implemented in a computational engine) is used to estimate the travel time and other performance measures associated with each scenario (TRB 2010a). (For information about default factors for urban streets reliability methodology, see Appendix H.)

The sequence of calculations in the reliability methodology is shown in Figure ES.4. The process is based on the urban streets engine. It begins with one or more engine input data files. An input file is modified during the scenario generation stage to reflect demand variation and the effect of other causes of nonrecurring congestion on running speed and saturation flow rate, as they occur during the reliability reporting period.

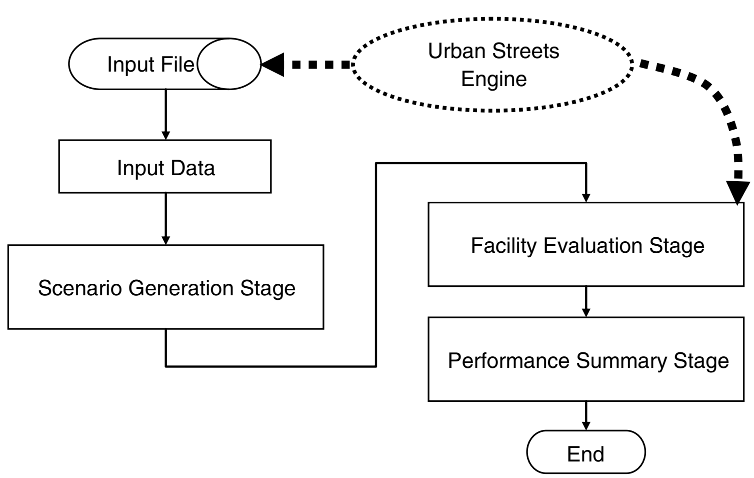


Figure ES.4. Urban streets methodology components.

Once all of the scenarios associated with the reliability reporting period have been generated, they are evaluated during the facility evaluation stage. The urban streets computational engine is used to automate the calculations. The evaluation results are then summarized during the performance summary stage. Various travel time distribution statistics and reliability performance measures are calculated for through vehicles traveling along the facility.

The freeway facility and urban street facility reliability models use different methods to develop a travel time distribution for the reliability reporting period:

- The freeway facility method develops scenarios on the basis of their probability of occurrence during the reliability reporting period. Some highly unlikely scenarios may be dropped from the analysis.
- The urban streets method randomly assigns demand, weather, and incident conditions to each day, on the basis of distributions of conditions likely to occur within a month. Some highly unlikely combinations may be included by random chance; therefore, multiple runs of the method may be needed to establish a representative travel time distribution.

Importantly, no direct link exists between the two methods. The weather pattern generated by the urban streets method may produce more or less severe conditions over a given model run compared with the 10-year average weather conditions used by the freeway method. An incident scenario for the freeway does not generate a corresponding high-demand scenario for the urban street. When local data are used to generate demand patterns, traffic diversion effects will appear in individual days' demands used to create month-of-day factors; but the effects of days with diversion will likely be washed out by demands from all of the days without diversion. When default demand pattern data are used, there is no diversion effect at all beyond that resulting from bad weather (and associated higher incident rates) occurring more often in some months of the year than in others.

Development of Scenario Generators

Scenario Generation for Freeway Facilities

A deterministic approach to scenario generation is proposed for freeway facilities. This deterministic approach enumerates different operational conditions of a freeway facility on the basis of different combinations of factors that affect travel time. These operational conditions are expressed as operational scenarios or, simply, scenarios. Four principal steps explain the construction of the scenario generation process for freeway facility analysis, as depicted in Figure ES.5.

The three main contributors to travel time variability on a freeway facility are variable demand level, weather, and incidents. Further, user-defined effects of work zones and special events can be incorporated as scenarios in the reliability analysis. These factors introduce stochasticity to

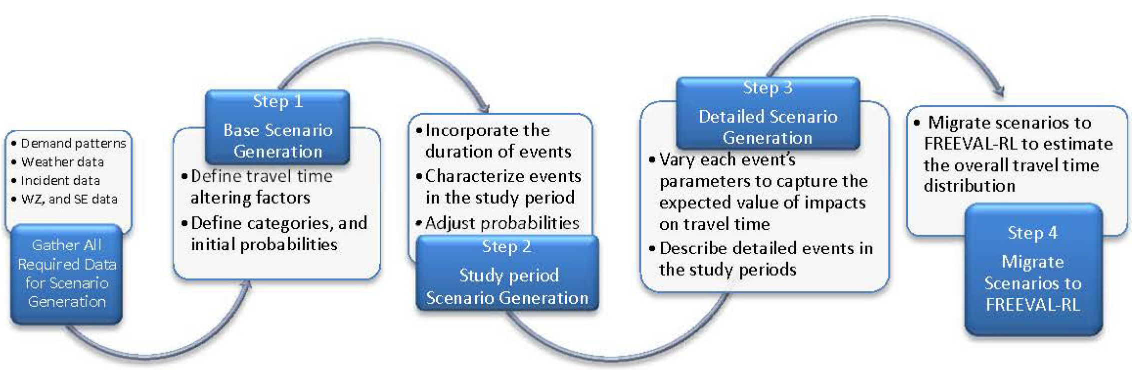


Figure ES.5. Process flow overview for freeway scenario generation.

travel time. In other words, they generate a travel time distribution instead of a deterministic and fixed travel time, as would be obtained by running a single study period. The reliability of a freeway facility is expressed as the portion of time in which the facility operates at or above the reliability standard set by the implementing agency.

The freeway scenario generation process uses a deterministic approach to model these variations. It categorizes different sources of variability (e.g., demand patterns or incident types) into different subcategories. For instance, weather—which is one of the main contributors to travel time variability—is defined in 11 weather categories (e.g., normal weather, medium rain, snow). Each category has a time-wise probability of occurrence and an impact on facility capacity, speed, and possibly demand. Thus, while the resulting distribution of travel times is stochastic, the process for generating scenarios is not; rather, it takes the approach of enumerating (nearly) all viable scenarios, each associated with varying probabilities of occurrence.

The mathematical performance model starts from the development of base, study period, and detailed scenarios. The latter are forwarded to the computational engine FREEVAL-RL for estimating analysis period facility travel times. While full automation has yet to be accomplished, the process readily lends itself to automation.

Scenario Generation for Urban Street Facilities

The scenario generation for urban streets consists of four sequential procedures. Each procedure processes the set of analysis periods in chronologic order.

- The first procedure predicts weather event date, time, type (i.e., rain or snow), and duration.
- The second procedure identifies the appropriate traffic volume adjustment factors for each date and time during the reliability reporting period.
- The third procedure predicts incident event date, time, and duration. It also determines incident event type (i.e., crash or noncrash), severity level, and location on the facility.
- The fourth procedure uses the results from the preceding three procedures to develop one urban streets engine input file for each scenario in the reliability reporting period.

Enhancements to the HCM Base Methodologies

Freeway Facilities Enhancements

The adaptation of the freeway facilities method developed by SHRP 2 Project L08 for performing a reliability analysis required several changes and enhancements to make the HCM methodology and associated computational engine "reliability ready." The enhanced computational engine is named FREEVAL-RL. The list of enhancements is as follows:

- Incorporation of the two-capacity phenomenon under queue discharge conditions;
- Improved modeling of capacity adjustment factors (CAFs) and speed adjustment factors (SAFs) for basic, merge, diverge, and weaving segments;
- New default values for CAF and SAF for incident and weather events on freeways;
- Enhanced performance measures for congested conditions; and
- Automation of computations.

The freeway reliability methodology can generate several thousand scenarios, many of which may have exceptionally low or exactly zero probability. In addition, some scenarios may be infeasible. The infeasible scenarios are automatically filtered out by the freeway scenario generation procedure. The scenarios with extremely low probability are not expected to be observed in the field in a single year; however, they are included in the predicted travel time index (TTI) distribution. This makes the comparison of predicted and observed distributions hard to interpret. In

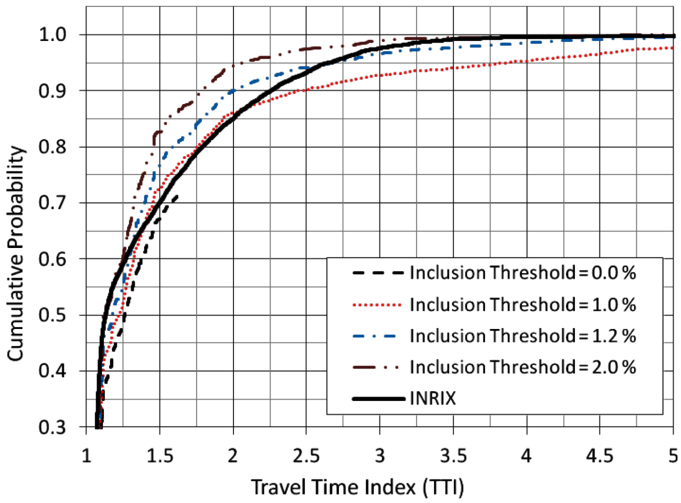


Figure ES.6. Predicted freeway HCM TTI distribution with different inclusion thresholds versus INRIX for 2010.

addition, these scenarios tend to have exceptionally large TTI values that significantly shift the tail of the cumulative distribution to the right (i.e., toward higher TTI values). Finally, these scenarios may also result in demand shifts in the real world that are not directly accounted for in the freeway reliability method.

To address these differences between predicted and observed distributions, the procedure allows the user to specify an "inclusion threshold" to include only scenarios with probabilities larger than the threshold specified for the analysis. For instance, an inclusion threshold of 1.0% means that only the scenarios with probabilities larger than 0.01 are considered in the analysis. Figure ES.6 presents the TTI cumulative distributions for four different inclusion threshold values for the case study of I-40 in Raleigh, North Carolina, and compares them with the observed TTI distribution obtained from the INRIX.com data warehouse.

Urban Street Enhancements

Three enhancements were made to the HCM2010 urban streets methodology. The first is a procedure for adjusting the discharge rate from a signalized intersection when a downstream incident or work zone blocks one or more lanes on the segment. The second is a procedure for computing the effective average vehicle spacing on a segment with spillback. The third is a methodology for using the HCM methodology to evaluate urban street facilities with spillback in one or both travel directions on one or more segments.

Validation of the enhanced methodology was based on a comparison of performance estimates obtained from a traffic simulation model. Three street segments were selected for the evaluation. The findings from this activity indicated that the enhanced methodology was able to provide accurate estimates of delay during congested and uncongested conditions.

Corridor Applications

A corridor study, by definition, goes beyond the single-facility focus of a typical HCM facility analysis. The purpose of a corridor study is to assess the ability of a subsystem of interrelated facilities to achieve a set of transportation performance objectives. For the purposes of a reliability analysis, *corridor* is defined as a freeway facility and one or more parallel urban street facilities. When traffic diversion occurs between the facilities in a corridor, the freeways, highways, and urban streets that cross the corridor and provide connections between the corridor's facilities will also be affected; however, those effects are beyond the scope of a reliability analysis. The focus of a corridor evaluation is on the parallel facilities.

An analysis of overall corridor reliability involves comparing selected reliability performance measures (e.g., travel time index, planning time index, percentage of on-time arrivals) generated for the individual facilities against either an established standard or against comparative national values of reliability. Because different agencies may be responsible for different facilities within a corridor (or, in the case of urban streets, different portions of the facility), and because corridor analysis focuses on longer-distance travel, a regional standard might be most appropriate. In the absence of such a standard, a percentile threshold could be used. In that case, unacceptable performance could be defined in terms of, say, a facility's planning time index (PTI) (e.g., among the worst 20% of U.S. facilities).

Potential Methods for Defining Level of Service by Using Reliability as a Service Measure

The research team initially considered four options as potential methods for defining level of service (LOS) by using reliability. Briefly, the options are as follows:

- *Reliability LOS based on current LOS ranges.* This option is the most consistent with current LOS concepts in the HCM. Inherently, a reliability analysis captures a range of operating conditions on the same facility, which are attributed to the various sources of (un)reliability. Using a distribution of LOS values therefore intrinsically mirrors the variability of traffic conditions on the facility.
- *Freeway reliability LOS based on travel speed ranges.* This option makes freeway reliability LOS conceptually consistent with urban streets and urban street segments. The problem of presenting a distribution rather than a single LOS value is still present.
- *Freeway reliability LOS based on most-restrictive conditions.* This option avoids the problem of presenting a distribution and assigns a single LOS value. It is more complicated to apply and explain in that two values must be set: a percentage threshold for the trips that fail to meet the LOS criteria and the ranges for each LOS category.
- *Reliability LOS based on the value of travel.* This option is the most complicated both to develop and explain. It has the advantage of being based on travelers' perception of reliability, but it relies on a factor (the reliability ratio, used to measure how travelers value reliability) that has not been precisely identified and will likely change with new research. Not only is this option complex, but establishing LOS ranges based on travel time equivalents is highly problematic.

Testing the four options with field data failed to reveal a clear choice on which to base reliability LOS. Furthermore, the four options were thought to be difficult to communicate to the profession, the public, and decision makers. As a result, the research team decided to develop an on-time–based measure, similar to Option 2. This measure, termed the reliability rating, is the percentage of trips serviced at or below a threshold TTI (the ratio of the actual travel time to the free-flow travel time). The TTI thresholds selected were 1.33 for freeways and 2.50 for urban streets. These thresholds approximate the points at which most travelers would consider a facility congested; thus, the measure roughly reflects the percentage of trips on a facility that experience conditions better than level of service F (LOS F). The difference in threshold TTI values results from differences in how free-flow speed is defined for freeways compared with urban streets, as TTI is measured relative to free-flow speed.

The research team did not define a service measure for travel time reliability. Because travel time reliability is a new concept for the transportation profession, the research team recommends that performance measures be used to describe the travel time reliability performance on freeways and urban streets. Subsequently, consideration can be given to using travel time reliability to define level of service. When reliability is considered as a service measure, the research team recommends that the reliability rating (now a performance measure) be the basis.

Other considerations for future reliability LOS deliberations are as follows:

- *Urban streets.* Figure 16-4 of HCM2010 defines LOS F as either (1) where the travel speed is 30% or less of the base free-flow speed or (2) where the subject through movement at one or more intersections has a volume-to-capacity ratio greater than 1.0 (TRB 2010a). Because the LOS definition is based on travel speed, which is a derivative of travel time, no changes in the LOS concept for urban streets is needed.
- *Freeways.* For freeway reliability, the research team first recommends that the existing density-based LOS definition be replaced with a travel speed–based definition. Density should be maintained as the indicator of general freeway performance, especially for rural facilities. The research team recommends that, in the future, travel speed be considered as a replacement to density even for general performance on urban facilities. The use of travel speed as the indicator of both general and reliability performance on freeways also provides consistency with the urban streets method. (See Appendix I for an example of existing freeway reliability.)

Future Research Needs

Supporting Implementation of the SHRP 2 Project L08 Research

The proposed HCM reliability chapters and the FREEVAL and STREETVAL software computational engines were completed in 2012 and reviewed by the TRB Highway Capacity and Quality of Service Committee in conjunction with the 2013 TRB annual meeting.

The computational engines consist of spreadsheets with embedded Visual Basic code. Separate Excel spreadsheet tools are used for generating the scenarios and then running the FREEVAL and STREETVAL engines to execute the HCM calculations in an automated fashion and process the results for reliability reporting purposes. While not part of the L08 project, a natural extension of the computational engines and other tools would be the development of a more user-friendly, integrated software tool that would execute the files faster than the Excel-based computational engines. Such a software tool could be hosted on a fast server and could be located in any secure environment, including a cloud-based environment. At present, the updated FREEVAL and new STREETVAL computational engines are hosted in the developer's environment at the contractor's site.

Freeway Facility Research Needs

Research needs in the freeway facilities methodology incorporate improvements to the core HCM methodology and to the submodels developed in the course of this study.

Research to Overcome Methodology Limitations

Although the research team was able to improve and expand the freeway facility methodology significantly during the course of this study, additional research is still needed to fill some gaps:

- The oversaturated flow-density relationship has not been calibrated since its inception in HCM2000.
- The spillback from off-ramps is not considered in the current methodology, significantly weakening its ability to model congested corridors.
- The free-flow speed and capacity adjustment factors used throughout the methodology to account for nonrecurring congestion effects have been adopted from the most recent and relevant literature and have not been locally calibrated or validated.
- The methodology does not include the effect of managed lanes on reliability.

Research to Improve the Reliability Submodels

Research is needed to understand and quantify the impact of weather, work zones, or special events on traffic demand:

- The method assumes that incident rates and weather conditions are independent. Research is needed to develop models that can explain the relationship.
- The current methodology does not account for weather events that have a small effect on segment capacity reduction (<4%). In addition, a given weather event (e.g., rain, snow) is always assumed to occur at its mean duration value. Furthermore, only two possible start times for weather events are considered.
- To consider the average effect of incidents on a facility, an incident is modeled only on three possible segments: the first segment, the segment at the facility midpoint, and the last segment. The timing of the incident is either at the start of a study period or at its midpoint. Finally, only three possible incident durations are considered: the 25th, 50th, and 75th percentiles of the incident duration distribution.

Urban Streets Research Needs

Future urban streets research is divided into two categories. The first category describes the research needed to overcome known limitations in the scope of the urban streets reliability methodology. The second category describes research needed to improve specific models within the reliability methodology.

Research to Overcome Methodology Limitations

In general, the urban streets reliability methodology can be used to evaluate the performance of most urban street facilities. However, the methodology does not address some events or conditions that occur on some streets and influence their operation. These events and conditions are identified as follows:

- Facilitywide performance measures;
- Truck loading and delivery;
- Signal malfunction;
- Railroad crossing and preemption; and
- Adverse weather conditions.

Research to Improve Specific Models

The urban streets reliability methodology was developed using currently available data and research publications. The data were used to calibrate the various models that make up the methodology. Calibration data were also collected in the field when existing data were not available. In some instances, the research team noted that a model's reliability could be improved if additional data were collected or made available through subsequent research. The following list identifies the specific models that would benefit from additional research:

- Wet-pavement duration;
- Effect of weather on signalized intersection saturation flow rate;
- Effect of incident length on segment operation; and
- Incident distribution.

CHAPTER 1

Introduction

This final report documents the activities performed during Phase 1 and Phase 2 of SHRP 2 Project L08, Incorporating Travel Time Reliability into the *Highway Capacity Manual*. The final report articulates the *how* and *why* of key decisions made and key activities undertaken during the project so that the logic and rationale are not lost to future researchers and practitioners who aim to build on the work completed in this effort.

Research Problem Statement

The *Highway Capacity Manual* (HCM) historically has been among the most important reference guides used by transportation professionals seeking a systematic basis for evaluating the capacity, level of service, and performance measures for elements of the surface transportation system—particularly highways, but also other modes. The HCM is useful for planning, design, preliminary engineering, and operations analysis. The manual provides analytic concepts for characterizing traffic flow, capacity, and quality and level of service. It also provides guidance on analyzing facilities, segments, and points for uninterrupted-flow roadways such as freeways and multilane highways, and for interrupted-flow roadway elements such as urban streets, signalized intersections, and two-way stop controlled intersections.

The HCM distinguishes between capacity and other performance measures. *Capacity* is defined as the hourly flow rate at which persons or vehicles can be reasonably expected to traverse a uniform section of road. Other performance measures include density, speed, delay, speed, number of stops, queue length, and volume-to-capacity ratio.

Travel time reliability is increasingly recognized as an important mobility performance measure. The HCM does not include a method to address travel time reliability. Nor does it have mobility performance measures or a method to address reliability for specific types of facilities such as freeways, multilane highways and urban corridors, and segments such as freeway weaving areas.

The HCM has undergone numerous updates since the first version was published in 1950; the most recent iteration is the HCM2010 (TRB 2010a). Much research has been completed within SHRP 2 that provides analytic procedures for computing travel time reliability on urban freeways. These analytic procedures are not in a form that can be applied directly to perform the types of analysis in the HCM. Moreover, gaps exist in SHRP 2 research regarding arterials and corridors. Nevertheless, the SHRP 2 Reliability research is a strong foundation for performing the type of analysis in the HCM to address nonrecurring congestion. In sum, analytic procedures are needed to incorporate travel time reliability into the methods used within the HCM.

Project Objectives

The main objective of this project is to determine how data and information on the impacts of differing causes of nonrecurrent congestion (e.g., incidents, weather, work zones, special events) in the context of highway capacity can be incorporated into the performance measure estimation procedures contained in the HCM2010. The methodologies in the HCM2010 for predicting delay, speed, queuing, and other performance measures for alternative highway designs are not currently sensitive to traffic management techniques and other operation/design measures for reducing nonrecurrent congestion. By including the effects of incidents, inclement weather, work zones, and demand variability on congestion and reliability, the methodologies produced by the L08 project are sensitive to many traffic management strategies, especially traffic incident management.

A further objective is to develop methodologies to predict travel time reliability on selected types of facilities and within corridors. Specifically, to do the following:

- Develop travel time reliability as a performance measure in the HCM2010 for freeway facilities;

- Develop travel time reliability as a performance measure in the HCM2010 for urban street facilities; and
- Address freeway and urban streets in a corridor context.

These procedures should inform planning, preliminary engineering, design, and systems operations and management.

Project Tasks

The following tasks were conducted as part of SHRP 2 Project L08.

Phase 1

Task 1. Conduct a literature review including but not limited to the HCM2010, SHRP 2 research (especially Projects L03 and L07 and other projects in both the Reliability and Capacity focus areas), state procedures, Federal Highway Administration (FHWA) source materials, and international input.

Task 2. Identify gaps in the availability of methodologies needed to satisfy the project objectives.

Task 3. Develop a study methodology to satisfy the project objectives. Include a definition of travel time reliability as a performance measure within the context of the HCM2010. Define and interpret key concepts of the proposed analytic and predictive procedures. Address reliability during different time periods. Ensure that the reliability performance measure is field-measurable.

Task 4. Develop a data collection plan and methodology that include data fusion requirements, quality assurance, testing, and validation.

Task 5. Prepare a Phase 1 report that includes a work plan for Phase 2. Present the results of Phase 1 to the Highway Capacity and Quality of Service Committee at a suitable location (a webinar may be used). The contractor may not proceed with Phase 2 until approval is received from SHRP 2.

Phase 2

Task 6. Collect data in accordance with the data collection plan/methodology.

Task 7. Analyze the data by following the methodology described in Task 3. Develop models that can be used to assess travel time reliability as a performance measure in the HCM2010 for (a) freeway facilities, (b) urban street facilities, and (c) freeways and urban streets in a corridor context.

Task 8. Test and validate models in a manner consistent with the methodologies established in Tasks 3 and 4.

Task 9. Develop computational procedures to document each model and make it easy to apply the reliability assessment models independently.

Task 10. Present two webinars during the course of the project, one before data collection and the other after preliminary model development. Record comments and questions from the webinar audiences and consider whether any are useful to the project.

Task 11. Prepare a guide that encompasses chapters evocative of the HCM2010 and addresses freeway facilities, urban street facilities, and freeways and urban streets in a corridor context. The text should be suitable for potential inclusion in a future update of the HCM.

CHAPTER 2

Defining and Measuring Reliability

Definitions for Reliability

Reliability is defined in engineering applications as "the probability that a component part, equipment, or system will satisfactorily perform its intended function under given circumstances." In statistics, *reliability* is defined as "the amount of credence placed in a result. The precision of a measurement, as measured by the variance of repeated measurements of the same object" (Parker 2003).

The Future Strategic Highway Research Program (F-SHRP) defined *travel time reliability* as the variation in travel times over time (e.g., hour-to-hour, day-to-day) (Cambridge Systematics, Inc. et al. 2003). SHRP 2 Project L03, Analytical Procedures for Determining the Impacts of Reliability Mitigation Strategies, defined the term as "the level of consistency in travel conditions over time, [which] is measured by describing the distribution of travel times that occur over a substantial period of time" (Cambridge Systematics Inc. et al. 2013). Other SHRP 2 projects also used the concept of variability to define reliability.

Given the wide range of viewpoints on what travel time reliability should encompass, the HCM2010 should have a broad definition of reliability. The following definition of travel time reliability is proposed:

Travel time reliability aims to quantify the variation of travel time. It is defined using the entire range of travel times for a given trip, for a selected time period (e.g., the p.m. peak hour during weekdays) over a selected horizon (e.g., a year). For the purpose of measuring reliability, a *trip* can be defined as occurring on a specific segment, facility (combination of multiple consecutive segments), or any subset of the transportation network; or the definition can be broadened to include a traveler's initial origin and final destination. Measuring travel time reliability requires that a sufficient history of travel times be present to track travel time performance. This history is described by the travel time distribution for a given trip.

Once the travel time distribution is established, several performance measures can be established to capture reliability. The two general types of reliability performance measures are the following:

1. Those that capture the variability in travel times that occurs for a trip over the course of time; and
2. Those that reflect the number of trips that either "fail" or "succeed" according to a predetermined performance standard or schedule.

In both cases, reliability (more appropriately, unreliability) is caused by the interaction of the factors that influence travel times: fluctuations in demand (which may result from daily or seasonal variation, or special events), traffic control devices, traffic incidents, inclement weather, work zones, and physical capacity (based on prevailing geometrics and traffic patterns). These factors produce travel times that vary from day to day for the same trip.

Both types of reliability measures are quantified from the distribution of travel times, for a given facility or trip and the time period (e.g., weekday peak period), which occurs over a significant span of time. One year is generally long enough to capture nearly all of the variability caused by disruptions. A variety of metrics can be computed once the travel time distribution has been established, including standard statistical measures (e.g., standard deviation, kurtosis), percentile-based measures (e.g., 95th percentile travel time, buffer index), on-time measures (e.g., percentage of trips completed within a travel time threshold), and failure measures (e.g., percentage of trips that exceed a travel time threshold). The reliability of a facility or trip can be reported for different slices of time, such as weekday peak hour, weekday peak period, or weekend.

Whether performance is being measured or predicted, uncertainty will intrude into the estimation of performance. In statistics, *uncertainty* is defined as "the estimated amount . . . by which an observed or calculated value may differ from the true value."

For the L08 project, no attempt was made to isolate the effects of measurement error or prediction error from the reliability measurements or estimates. The separate effects of measurement uncertainty on the reliability data sets were not accounted for. Similarly, when dealing with predictions of performance, no attempt was made to add a separate component for prediction uncertainty. Therefore, this project's measurements of reliability include measurement uncertainty and its predictions of reliability exclude prediction uncertainty. To the extent possible, analysts should compare model results with the performance of several facilities in their area by using locally developed inputs to gain an idea of the prediction uncertainty.

Terminology

The following terminology from the HCM2010 is used in this report:

- *Analysis period* is the smallest time unit for which the HCM analysis procedure is applied. In the case of freeway and urban street facility analysis, the HCM analysis period is 15 min, although it can be of greater of duration, at the discretion of the analyst. Alternative tools may define different analysis period lengths.
- *Study period* is the sum of the sequential analysis periods for which the HCM facility analysis procedure is applied (e.g., a 4-hour peak period). The study period is defined by the analyst for each specific application, on the basis of the guidance provided in the HCM.

For the purposes of the L08 research, the following additional term is used:

- *Reliability reporting period* is the period over which reliability is to be estimated (e.g., the 250 nonholiday weekdays in a year). In essence, the reliability reporting period specifies the number of days for which the reliability analysis is to be performed.

The three terms are illustrated in Figure 2.1.

Reliability Metrics

A variety of measurement and modeling methods have been used to calculate travel time, which is the basis for reliability. In their purest form, travel times are directly measured as the time it takes vehicles to traverse a highway section with known or fixed endpoints. Excepting manual methods, this may be done with roadway or vehicle-based detection methods. In the roadway method, equipment placed at the endpoints detects the times that an individual vehicle passes the points. Several technologies can be used to detect vehicles passing a point, including toll tag readers, electronic license plate readers, vehicle signature recognition, and interception of signals from on-board electronic devices (e.g., Bluetooth). Vehicle-based methods require that equipment on the vehicle be capable of detecting and transmitting the vehicle's time and location; this is usually done using global positioning system (GPS) technologies.

Calculation of Travel Time

Roadway and vehicle-based methods are the most accurate for measuring travel time because they are direct measurements. An indirect method of measuring travel times that is in widespread use is to use spot measurements of speeds from roadway detectors on uninterrupted-flow facilities; volumes and loop occupancies are usually measured as well. In this method, which relies on a series of relatively closely spaced (½ mile or less) roadway detectors, the spot speed measurement (generally

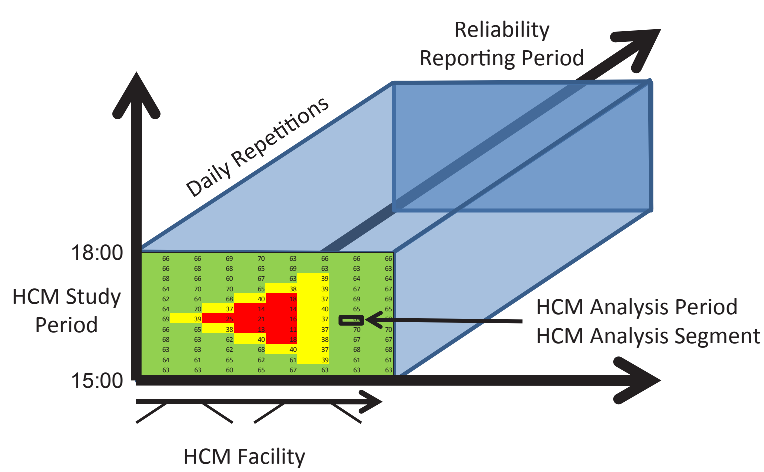


Figure 2.1. Study facility and period, and analysis segment and period.

considered to be a time mean speed) from a detector is assumed to be constant over a fixed distance (e.g., half the distance to the next upstream and downstream detectors). If that distance is known, a travel time can be computed from the assumed speed and length.

As already stated, the distribution of travel times is the starting point for measuring reliability. In a statistical sense, the distribution is continuous only if it is based on measuring travel times from individual vehicles. At the time of writing, the data used to monitor travel times—as well as modeling methods—are rarely managed in this way. For example, consider roadway detectors of spot speeds, which measure every vehicle that crosses their detection zone. These systems are designed to aggregate field measurements into 20- or 30-s summaries before transmission. Therefore, in its lowest form, the speed "measurement" is really an average. The data are sometimes further aggregated to 1-, 5-, or 15-min summaries for archiving. At each aggregation, variability in the measurements is reduced. (When aggregating travel times over analysis periods, it is important to weight the travel time averages by volume or vehicle miles traveled, VMT, rather than taking just the arithmetic mean.) On the other hand, roadway- and vehicle-based systems have a sampling rate well below 100%.

This discussion is relevant to this research. The macroscopic analysis engines used here—FREEVAL (FREeway EVALuation) and STREETVAL (STREET eVALuation)—are not intended to produce travel times for individual vehicles. Instead, they produce an estimate of the mean travel time for each time slice studied, set at 15-min intervals by the HCM. Therefore, some variability is not accounted for in the analysis. The basic unit of measurement used to construct the travel time distribution, from which reliability metrics emerge, is then a 15-min average. Likewise, if archived roadway spot speed detectors are used, the unit of measurement is also an average travel time for whatever aggregation level is used.

Does this loss of variability information matter? The answer depends on the viewpoint and use of the method. If travel times from individual vehicles are used, then the resulting reliability metrics will capture not only the effect of external sources (such as incidents and inclement weather) but the differences in driver behavior as well. Capturing the total amount of variation may be important for some applications. For practitioners, at least with current technologies, controlling driver behavior is not an option—driver "aggression" is not affected by the control strategies that can currently be implemented. For capturing the effect of the major sources of congestion and reliability, the effect on driver behavior may be ignored. If this is done, then the resulting documentation must state that the reliability statistics developed do not account for differences in driver behavior. This is the case for the methods developed by this research. The research team emphasizes that, in interpreting the reliability metrics produced by HCM2010 methods, the estimate corresponds to the distribution of aggregated travel times (into 15-min bins), as opposed to the individual driver experience. In other words, the *mean* of the HCM-based travel time distribution really is a mean of 15-min averages, as opposed to a (true) mean of individual travel time observations.

Recommended Reliability Metrics

As a starting point, the reliability metrics developed in SHRP 2 Project L03 (Cambridge Systematics, Inc. et al. 2013) were reviewed for relevancy to the HCM. A discussion of those measures follows. Metrics that describe the right half of the travel time distribution are the most appropriate for reliability, because that is the region in which the causes of unreliable travel (disruptions and high demand) exert the most influence.

The reliability rating is the percentage of trips experiencing a travel time index (TTI) less than 1.33 for freeways and 2.50 for urban streets. (The TTI is the travel time divided by the free-flow travel time.) The selected thresholds approximate the points at which most travelers would consider a facility congested; thus, the measure reflects the percentage of trips on a facility that experience conditions better than level of service F (LOS F). The difference in threshold TTI values results from differences in how the HCM defines free-flow speed for freeways versus urban streets, as TTI is measured relative to free-flow speed.

The planning time index (PTI) and buffer index are starting to be used in practice, primarily for performance monitoring applications. The PTI is the 95th percentile travel time divided by the free-flow travel time, while the buffer index is the 95th percentile travel time divided by the mean or median travel time. SHRP 2 Project L03 found that the buffer index can be an unstable indicator of changes in reliability because it can move in a direction opposite to the mean and percentile-based measures. This occurs because it uses both the 95th percentile and the median or mean travel time, and the percentage change in those values can vary from year to year. Although not specifically tested during the L03 project, the skew statistic (the ratio of the difference between the 90th and 50th percentile TTIs and the difference between the 50th and 10th percentile TTIs) may also suffer from this phenomenon. These observations led the research team to the conclusion that L08 reliability metrics should be ones that are measured relative to the free-flow travel time. Metrics that are measured relative to parameters that can change (e.g., the mean or median) are not constant over multiple 15-min time intervals. They are therefore more difficult to quantify across an extended time-space domain.

The 80th percentile TTI has not been widely used. However, SHRP 2 Project L03 found this measure to be more sensitive to operational changes than the 95th percentile TTI and recommended its use. Furthermore, one of the more reliable past studies of reliability valuation used the difference between the 80th and 50th percentile travel times as the indicator of reliability.

The misery index, the average of the highest 5% of travel times, approximates the 97.5 percentile TTI. This measure is useful as a descriptor of near-worst-case conditions on rural facilities.

Standard deviation was not part of the L03 set of measures, but it should be added because of its use in applications. SHRP 2 Projects C04 and L04 use standard deviation as one of the terms in expanded utility functions that are used to predict traveler behavior. Several studies of reliability valuation have used standard deviation as the measure that is valued.

Failure and on-time measures are defined in two ways: (1) in reference to the median travel time (used to indicate "typical" conditions for a trip) and (2) in relation to predetermined performance standards based on the space mean speed (SMS) of the trip. Because their construction is binary (a trip either passes or fails the condition), these measures can be insensitive to small changes in underlying performance. Therefore, they have been defined with multiple thresholds so that changes in performance can be more easily detected. The median-based measures are constructed as on-time measures, while the SMS measures are constructed as failure measures.

SHRP 2 Project L02 investigated two other metrics—the semivariance and its companion, the semistandard deviation—for measuring reliability. These are computed similarly to the typical variance and standard deviation, except they pertain only to observations on one side of a reference value. (The variance and standard deviation measure both sides of a reference value, which is the mean.) Project L02 selected the free-flow travel time as the reference value. The calculation of the semivariance is then the sum of the squared differences between observed travel times and the free-flow travel time, divided by the number of observations. The semistandard deviation is the square root of the semivariance. It is assumed that the free-flow travel time is the minimum travel time for the section. In practice, high-speed vehicles lead to lower travel times than that for free flow, but for consistency in measurement, the free-flow travel time is used. Project L02 found the semivariance to be a stable indicator of variation across multiple types of distributions. The L08 research team recommends adding the semistandard deviation as a reliability performance metric.

In many cases, an analyst may wish to evaluate several of these measures to obtain the most complete picture of travel time reliability. However, as a single measure that reflects the traveler's point of view and LOS F conditions as defined in HCM2010 Chapters 10 and 16, the research team recommends reporting the reliability rating as part of any HCM-based reliability analysis.

On the basis of this discussion, the metrics in Table 2.1 are recommended for Project L08. Both variability- and failure-based metrics are included. Which metric should be highlighted as the primary reliability metric is difficult to say. Much depends on the specific application being used. In the interpretation of Table 2.1, many of the selected performance measures are defined relative to the *free-flow* travel time, rather than the *average* travel time. This is deliberate because the average travel time (a) is not known before the analysis is conducted, (b) varies between different facilities, and (c) varies between different scenarios (e.g., advanced traffic demand management treatments) for the same facility. Performance measures based on the average travel time are therefore deemed to be less appropriate for HCM analysis and stratification of LOS.

Table 2.1. Recommended Reliability Performance Measures for SHRP 2 Project L08

Reliability Performance Measure	Definition
Core Measure	
Reliability rating	Percentage of trips serviced at or below a threshold travel time index (TTI) (1.33 for freeways, 2.50 for urban streets)
Planning time index (PTI)	95th percentile TTI (95th percentile travel time divided by the free-flow travel time)
80th percentile TTI	80th percentile TTI (80th percentile travel time divided by the free-flow travel time)
Semistandard deviation	The standard deviation of travel time pegged to free-flow travel time rather than the mean travel time (variation is measured relative to free-flow travel time)
Failure or on-time measures	Percentage of trips with space mean speed less than 50, 45, and/or 30 mph
Supplemental Measure	
Standard deviation	Usual statistical definition
Misery index (modified)	The average of the highest 5% of travel times divided by the free-flow travel time

CHAPTER 3

State of the Art and State of the Practice

Domestic and International Agency Usage

California

The California Department of Transportation (Caltrans) has produced performance measures for the entire multimodal system (Downey 2000). The measures are intended to do the following:

- Monitor and evaluate system performance.
- Share existing data and forecast future performance information.
- Develop mode-neutral customer and decision information.
- Build consensus using performance measures information.
- Improve accountability of system development and operations.

Caltrans tested the measures on corridors in four metropolitan counties in 2000. The peak period travel time varied by 10% to 50% on all corridors. The agency also found that reliability may not be directly correlated with delay; some areas that had high delay also had low travel time variability, partly because of the difficulty in deviating from slow speeds. Travel time reliability depended on several factors, including distance between interchanges and roadway geometrics.

Caltrans began to measure travel time reliability in January 2011. *Travel time reliability* is defined as the predicted mean travel time compared with the actual travel time. The geographic coverage of the measurement is evolving, and recent changes have focused on selected corridors. For each corridor, division and district transportation professionals calculate one or more reliability measures.

Florida

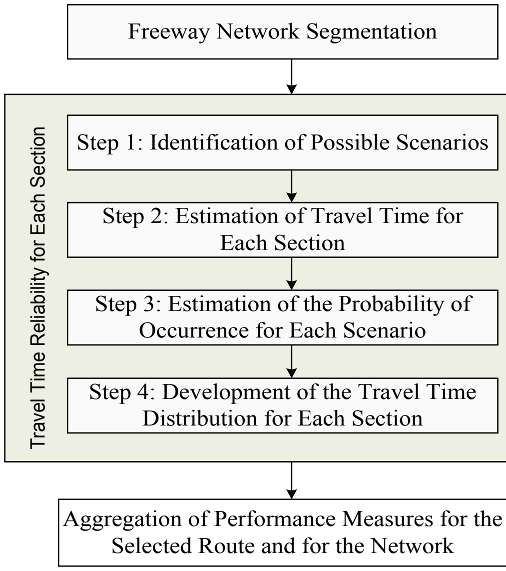
The Florida DOT (FDOT) has developed a method and tools for estimating travel time reliability for the freeway portion of its Strategic Intermodal System (SIS). The method is illustrated in Figure 3.1. As shown in the figure, the freeway network to be analyzed is first segmented at a section level (interchange to interchange). Interchanges and beginning and ending milepost numbers are obtained from FDOT's Roadway Characteristics Inventory.

As a first step in the travel time reliability estimation, the methodology considers a variety of possible scenarios that may occur on any given freeway section. These scenarios are based on the presence of congestion, rain, incidents, and work zones. For example, one scenario may be that the section is congested and an incident is occurring along its length. Another scenario may be that the section is not congested, but it has a work zone along its length.

In the second step, the method estimates the travel time for each scenario identified in Step 1. The travel time estimation is based on a combination of previously developed models (HCM2000; Elefteriadou et al. 2010a). In the third step, the method obtains the probability of occurrence for each scenario identified in Step 1. The fourth step develops the travel time distribution for the section and estimates selected travel time reliability measures on the basis of this distribution.

Finally, the travel time reliability for the entire freeway network is estimated by aggregating the respective measures for each of the sections analyzed. The travel times for each of the segments within a given route are summed for each hour to obtain facility travel times. From these, travel time reliability measures are calculated in a similar way to those for segments.

FDOT is currently proceeding with obtaining metrics for both categories of travel time reliability definitions (i.e., based on the traditional concept of reliability as nonfailure over time and based on the concept of variability of travel time). The FDOT Traffic Operations Office is interested in travel time variability, which it would ultimately like to report to travelers on a real-time basis; the Systems Planning and Policy Planning Offices are interested in the on-time arrival estimation and in the evaluation of the performance of the



Source: Elefteriadou et al. (2010a).

Figure 3.1. FDOT reliability methodology overview.

SIS so that improvements can be prioritized on the basis of this measure and reported to decision makers. Additional information regarding this method is provided in a series of reports (Elefteriadou and Xu 2007; Elefteriadou et al. 2008, 2010b, 2010c).

Current research work by FDOT focuses on the development of models for arterial sections of the SIS and on the refinement of existing freeway models by comparing their output with field data from instrumented sections.

Nevada

Nevada DOT's Integrated Transportation Reliability Program (ITRP) aims to implement new and innovative programs to prevent congestion and improve reliability. As part of the program, the agency will coordinate with statewide stakeholders to develop strategies to improve travel time reliability in Nevada.

More than 15,000 traffic crashes occur each year in the Las Vegas valley. The Las Vegas Traffic Incident Management Coalition brought southern Nevada emergency response and transportation agencies together to enhance emergency response. The group established collision clearance time goals to restore road travel following traffic crashes (Kimley-Horn and Associates, Inc. 2010).

International Research

This section draws from a range of experiences by international transportation agencies. Some of the information is drawn from an FHWA international scan of transportation performance measurement practices. The scan included visits to transportation agencies with mature performance management systems in Australia, Great Britain, New Zealand, and Sweden. It focused on how these organizations demonstrate accountability to elected officials and the public. One of the interests of the scan team was how transportation agencies used reliability performance measures and practices to meet their goals (Braceras et al. 2010).

All of the agencies reported that their reliability measures were evolving and they were not entirely satisfied with their measurement tools. However, the more urbanized agencies in Great Britain, Australia, and Sweden had clearly invested considerable effort in measuring real-time highway, transit, and rail operations to improve travel time reliability, enhance transportation choices, and reduce greenhouse gas emissions. This section describes several findings from the international scan, along with Japanese and Dutch research.

Great Britain

National

The British have invested considerable effort in measuring reliability on high-volume national routes. The Highways Agency (HA) of Great Britain has identified a Strategic Road Network of 2,700 km (1,678 mi) of motorways and 4,350 km (2,703 mi) of other trunk routes. These routes are analyzed in 103 sections with 2,500 total links. The HA actively tracks reliability performance on a daily basis across this network and defines travel time reliability as the average vehicle delay on the slowest 10% of the journeys (Cambridge Systematics, Inc. et al. 2013).

The network reliability program has improved British officials' understanding of system performance, and the HA has increased its use of reliability analysis in evaluating improvement strategies. The HA identified several difficulties in measuring reliability, including shortcomings in data and varying definitions. They also noted difficulties explaining the results to the public because the performance measures are not very sensitive to the improvements. For example, improvements reduced the average of the worst 10% of trips making a 16-km journey from 3.9 min to 3.4 min of delay. For a slow trip, an improvement of half a minute is marginal. Additionally, the HA could not be sure whether the improvement created the travel time reliability benefit or whether it was a function of changes in economic conditions.

London

Research found that travel time varies in three ways: interday variability (caused by seasonal and day-to-day variations in travel times), interperiod variability (caused by different departure times and consequent changes in congestion) and intervehicle variability (caused by personal driving styles and behavior of traffic signals along a certain route) (Bates et al. 1987). The authors measured travel time reliability using the mean-variance approach (based on variance or standard deviation of travel times), the scheduling approach (based on disutility incurred because of late arrivals), or the probabilistic/mean lateness approach (based on mean lateness at departure/arrival).

Sweden

The Swedish Road Administration (SRA) includes travel reliability among a large set of transportation performance measures. Travel times and speeds are tracked on major routes in the three major cities (Stockholm, Malmö, and Göteborg) and on routes to towns for rural residents (Franklin 2009). The SRA reports are designed to connect the performance of the system with "the steps taken in each area to improve traffic flow and reliability and report on planned improvement strategies for the next year." Rural reporting includes the effect of seasonal weather problems and summarizes the number of residents who saw increases or improvements in travel times between towns.

Japan

Use of predicted reliability within project benefit-cost analysis is in its nascent stages in Japan. Higatani et al. (2009) examined the characteristics of travel time reliability measures using traffic flow data from the Hanshin Expressway, an urban toll expressway network that stretches from Osaka to Kobe. For this study, travel time reliability indices were calculated for five radial routes connected to the downtown loop route in Osaka City. Several measures were calculated for one radial route (Route 11 Ikeda Line), including average travel time, 95th percentile travel time, standard deviation, coefficient of variation, buffer time, and buffer index. The buffer time and buffer index showed tendencies similar to the standard deviation and coefficient of variation, respectively. The time-of-day variation of traffic flow was also investigated for all five radial routes, and the effect of traffic incidents on travel time reliability measures was analyzed for one radial route (Route 14 Matsubara Line).

The Netherlands

Research found that travel time variance accounts for only a portion of the delay effects from unreliability. The studies recommend including the skew of travel time distribution (e.g., the amount of extra travel time for the worst 5% of trips) to measure the remaining effects of unreliable travel times (van Lint et al. 2008; Tu 2008).

U.S. Research

SHRP 2 Project L03

SHRP 2 Project L03 examined the potential performance measures used to describe travel time reliability. Table 3.1 summarizes the recommended reliability performance metrics from that study. The recommendations were based on an examination of measures in use in the United States and in

Table 3.1. Reliability Performance Metrics from SHRP 2 Project L03

Reliability Performance Metric	Definition	Units
Buffer index (BI)	The difference between the 95th percentile travel time and the average travel time, normalized by the average travel time The difference between the 95th percentile travel time and the median travel time, normalized by the median travel time	Percent
Failure or on-time measures	Percentage of trips with travel times less than 1.1 × median travel time and/or 1.25 × median travel time Percentage of trips with space mean speed less than 50, 45, and/or 30 mph	Percent
80th percentile TTI	80th percentile travel time divided by the free-flow travel time	None
Planning time index	95th percentile TTI (95th percentile travel time divided by the free-flow travel time)	None
Skew statistic	The ratio of (90th percentile travel time minus the median) divided by (the median minus the 10th percentile)	None
Misery index (modified)	The average of the highest 5% of travel times divided by the free-flow travel time	None
Standard deviation of travel time or travel rate[a]	Standard statistical definition	None

[a] Not included in the L03 recommendations, but added here. See text.
Source: Cambridge Systematics, Inc. et al. (2013).

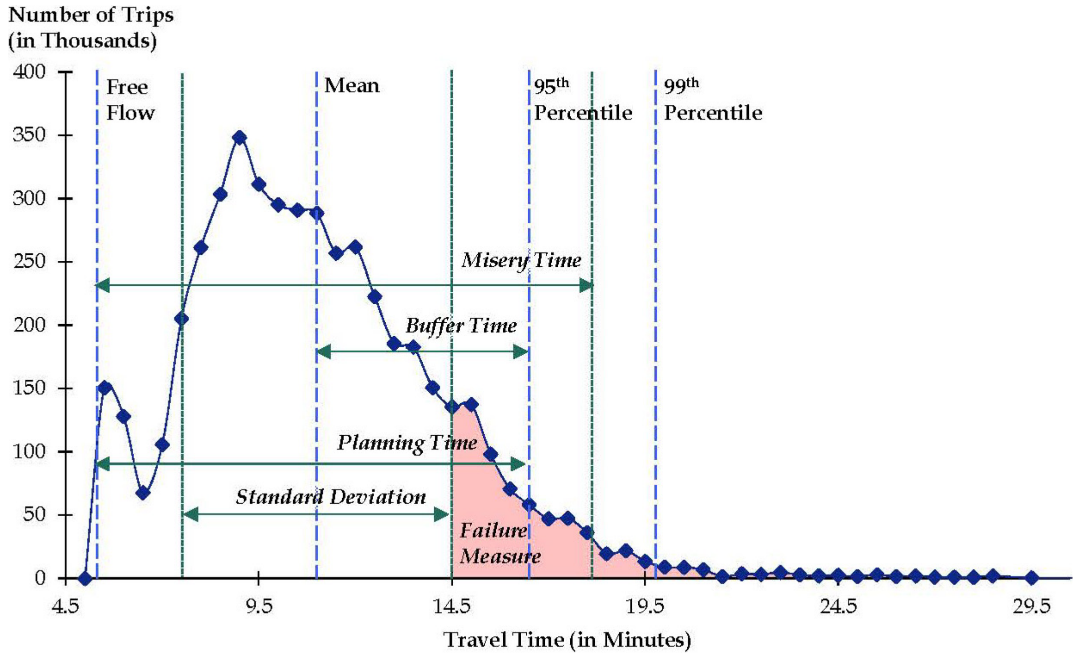


Figure 3.2. Travel time distribution as the basis for defining reliability metrics.

other parts of the world. The table also includes the skew statistic proposed by European researchers. In addition, the researchers added the 80th percentile TTI because analysis indicated that this measure is especially sensitive to operations improvements, and it has been used in previous studies on the valuation of reliability. All of these measures can be easily created once the travel time distribution is established, as illustrated in Figure 3.2. Because of the need to normalize travel time, the TTI was used as the variable of interest in this research. Therefore, the base distribution is actually based on the distribution of the TTI, rather than raw travel times.

The L03 research also demonstrated that the buffer index can be an unstable measurement for tracking trends over time in part because of its linkage to two factors that change (average and 95th percentile travel times); if one changes more in relation to the other, counterintuitive results can appear.

Note that standard deviation of travel time or travel rate appears in Table 3.1 and Figure 3.2. Project L03 did not define this as a reliability performance metric, but it has been added because several other SHRP 2 research projects have indicated that it is useful in both costing reliability and in modeling traveler choices. Project L03 included predictive methods for the standard deviation, even though it was not formally identified as a useful performance measure because of the difficulty in explaining it to nontechnical audiences.

NCHRP Project 3-97

NCHRP Project 3-97, Traffic Signal Analysis with Varying Demands and Capacities, developed a recommended set of performance measures for evaluating the robustness of signal timing plans when challenged with varying demand and capacity conditions (Dowling et al. 2011). Robustness is defined as the ability of the signal system to continue to provide satisfactory performance under varying demand and capacity conditions.

Three measures were recommended for evaluating signal system performance. One relates to average performance, taking into account expected fluctuations in demand and saturation flow rates for the analysis period over an extended period. The other two relate to the robustness of the timing plan when challenged with demand and saturation flow rate fluctuations.

The first measure of effectiveness (MOE) is the weighted-average whole-year performance for the subject peak period. Performance can be measured using any one of many commonly used signal performance measures (e.g., delay, stops, performance index). Differences in the average performance between two peak period timing plans can be used to compute differences in total performance. For example, the difference in the average vehicle hours traveled multiplied by the number of nonholiday weekdays per year can be used to estimate total annual vehicle hours saved for one plan versus the other.

The second MOE is the 95th percentile performance (or a similar high-percentile performance). The analyst can interpret this MOE to mean that 95% of the time the performance of the timing plan will not be worse than the values associated with the 95% scenario.

The third recommended MOE is the probability of breakdown. The probability is that demand will exceed the timing plan capacity for the duration of the analysis period (typically

1 to 2 hours) somewhere in the system. When demand exceeds capacity for extended periods, queues and delays build rapidly. The probability is a quick, intuitive measure of the likelihood of capacity failure with the current plan.

FHWA ATDM Evaluation Guidebook

The FHWA's *Guide for Highway Capacity and Operations Analysis of Active Transportation and Demand Management Strategies,* which was in publication at the time of writing, recommends a basic set of reliability performance measures from which various statistics can be computed (Dowling and Margiotta 2013). The guidebook then recommends a specific set of measures of effectiveness that may be useful for comparing performance across different Active Transportation and Demand Management (ATDM) strategies and might eventually serve as a foundation for a level of service measure of reliability.

The basic performance measures are useful for most economic and environmental analyses. In addition, the basic performance measures are key components of the recommended measures of effectiveness for evaluating ATDM.

The recommended MOEs are designed to address two key objectives of ATDM: to improve facility/system efficiency and to improve reliability. In addition, two of the recommended MOEs provide measures that individuals can relate to: average speed and average delay per trip.

The recommended basic performance measures and measures of effectiveness for evaluating the performance benefits of ATDM measures are

- Basic performance measures useful for computing MOEs:
  - Vehicle miles traveled demand (VMT-Demand)
  - Vehicle miles traveled served (VMT-Served)
  - Vehicle hours traveled (VHT)
  - Vehicle hours delay (VHD).
- Measures of effectiveness:
  - System efficiency: average system speed (mph)
  - Traveler perspective: vehicle hours delay per vehicle trip (VHD/VT)
  - Reliability: planning time index (PTI).

The VMT-Demand is the sum of the products of the input origin–destination (O–D) table vehicle trips and the shortest-path distance between each origin and destination. Although not traditionally a performance measure for highway improvement projects, demand is a measure of the success of ATDM at managing the demand for the facility. The VMT-Served is the sum of the products of the total link volumes for the peak period and the link lengths. VMT-Served is a measure of the productivity of the facility, the improvement of which is one of the key objectives of ATDM.

VHT is the sum of the products of the total link volumes and the average link travel times. Delays to vehicles prevented from entering the facility during each time slice (vehicle hours of entry delay, VHED) (either by controls, such as ramp metering, or by congestion) are added to and included in the reported VHT total.

VHD is the difference between the VHT (including vehicle entry delay) and the theoretical VHT if all links could be traversed at the free-flow speed with no entry delays. VHD is summed over all time slices within the scenario. VHD is useful in determining the economic costs and benefits of ATDM measures. VHD highlights the delay component of system VHT.

$$\mathrm{VHD} = \mathrm{VHT} - \mathrm{VHT(FF)} \tag{3.1}$$

where

VHD = vehicle hours delay;

VHT = vehicle hours traveled, including vehicle entry delay; and

VHT(FF) = vehicle hours traveled, recomputed with segment free-flow speeds.

VHED for any given scenario is the number of vehicles prevented from entering the system during each time slice, multiplied by the duration of the time slice and summed over all time slices. VHED should be included in the computed VHD and VHT for each scenario.

Average system speed (mph) is a measure of the efficiency of the highway system. It is computed by summing the VMT-Served for each scenario, then dividing by the sum of the scenario VHTs (including any vehicle entry delay). One of the key objectives of ATDM is to maximize the productivity of the system, serving the greatest number of VMT at the least cost to travelers in terms of VHT. Thus, changes in the average system speed are a good overall indicator of the relative success of the ATDM strategy at achieving its objective of improving efficiency.

Vehicle hours delay per vehicle trip (VHD/VT) is the vehicle hours delay summed over all scenarios divided by the sum of the number of vehicle trips in the origin–destination (O-D) tables for all scenarios. This gives the average delay per vehicle, which is useful for conveying the results in a manner that can be related to personal experience.

The travel time index (TTI) is a measure of congestion on the facility. It is the ratio of the mean travel time to the free-flow travel time. For example, a TTI of 1.20 can be interpreted as meaning that the traveler must allow 20% extra time over free-flow travel time to get to his or her destination on time. When a percentile greater than 50% is used, then the TTI becomes a reliability measure. For example, an 80th percentile TTI of 1.20 can be interpreted as meaning that, over the course of a year for a given trip leaving at a given time, 80% of the trips will take no more than 20% longer than the free-flow travel time. A 95th percentile TTI is also referred to as the planning time index (PTI).

While various travel time percentiles historically have been used for the TTI, is the L08 team recommends that the 80th percentile highest travel time be used for the predicted travel time. The 80th percentile travel time has a more stable relationship to the mean travel time than the 90th, 95th, or 99th percentiles, so it is useful in predicting changes in reliability that are based on changes in the mean travel time. The formula for computing a systemwide TTI follows:

$$80\%\mathrm{TTI} = \frac{\mathrm{VHT}(80\%)/\mathrm{VMT}(80\%)}{\mathrm{VHT}(\mathrm{FF})/\mathrm{VMT}(\mathrm{FF})} \tag{3.2}$$

where

80%TTI = 80th percentile travel time index;

VHT(80%) = 80th percentile highest vehicle hours traveled among scenarios evaluated;

VMT(80%) = vehicle miles traveled for scenario with 80th percentile highest vehicle hours traveled among scenarios evaluated;

VHT(FF) = vehicle hours computed with segment free-flow speeds; and

VMT(FF) = vehicle miles traveled with segment free-flow speeds.

CHAPTER 4

Development of Freeway and Urban Streets Methodologies

The SHRP 2 Project L08 conceptual analysis framework (L08 framework) for predicting travel time reliability is designed for operations analysis and planning applications in which the analyst must estimate current reliability on the basis of a limited amount of data, or predict the impacts of demand changes, operational improvements, and design concepts on reliability. The L08 framework requires fewer analytical resources than the SHRP 2 Project L04 framework, which involves simulation models and demand models. And the L08 framework provides more detailed information on the sources of unreliability and the reliability effects of specific operational improvements than the SHRP 2 Project L03 regression equations. Thus, the L08 framework is designed to work with the existing HCM2010 methodologies for evaluating freeway and urban street facilities.

Overview of the L08 Conceptual Analysis Framework

The L08 framework employs scenarios to diagnose the causes of unreliable performance and to predict the impacts of specific operational improvements on reliability, as shown in Figure 4.1. Each scenario is a specific combination of demand, weather, incidents, special events, and work zones. Each scenario presents a challenge to the operation of the facility. HCM2010 methodologies are used to predict how facility performance responds to each of the challenges. Special capacity, saturation flow, and free-flow speed adjustment factors have been developed by the L08 team for use with the HCM2010 methods to account for the effects of weather and incidents on capacities and speeds. The HCM2010 methods themselves have also been selectively augmented to facilitate their application to reliability analysis. The results of the numerous challenges are then summed up and weighted according to their probability of occurrence to obtain statistics on the facility's reliability.

Freeway and Urban Streets Methodologies

While the overall reliability analysis framework is identical for freeway and urban street facilities, the specific implementations of the L08 framework vary between freeway and urban street facilities applications to suit the specific characteristics of each facility analysis methodology in the HCM2010.

The HCM2010 freeway analysis method deals with the study period and study section of the facility as a whole, breaking down the calculations to specific analysis periods and segments within the facility. The performance of all traffic movements sharing a mainline freeway segment is evaluated. Only one direction of flow on the freeway is evaluated at a time.

The HCM2010 urban streets method also deals with the study section of the facility as a whole (with disaggregation for segments and intersections), but it focuses on a single 15-min analysis period within the peak hour. The method can be used to build up analysis periods into study period results, but that is not its common use. The method evaluates performance only for the through movement on the arterial (taking into account the effects of other movements on the through movement performance) and evaluates both directions of travel at a time.

These differences in the HCM2010 freeway and urban streets methods have resulted in two implementations of the L08 framework that take two slightly different approaches to estimating reliability. The L08 freeway method requires the analyst to provide a full study period of 15-min demands and then apply monthly and daily factors to obtain demand variation over the reliability reporting period. The L08 urban streets method requires hourly demands, breaks those down into 15-min periods, and then extrapolates the hourly demand to the study period. Like the freeway method, the urban streets method also applies monthly and daily factors to obtain demand variation over the reliability reporting period. Both methods leave out some of the true day-to-day variability of demand by using average monthly and daily factors to generate their varying demands.

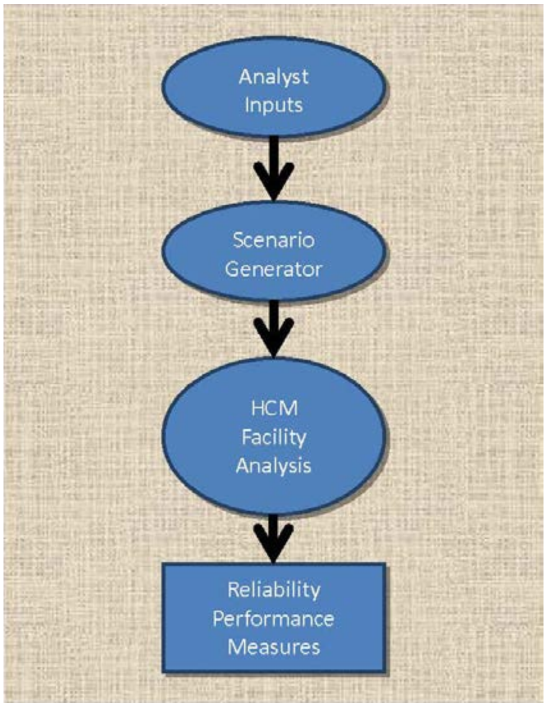


Figure 4.1. Conceptual analysis framework.

A more significant difference in the two implementations of the L08 framework is the use of stochasticity within the methods. The L08 freeway method is primarily a deterministic approach that applies probabilities at the end of the process when tallying the reliability statistics. The L08 urban streets method is primarily a stochastic approach that applies stochastic methods (random numbers) within the scenario generation process to generate one set out of many possible sets of scenarios for the street. Knowledge of the probabilities of the scenarios at the end is not required because that is built into the steps used to create the scenarios in the urban streets method. Thus, the freeway analysis can identify specific scenarios with relative ease, while the number of potential scenarios for urban streets is very large (e.g., turning percentages, signalization, location of incidents). Therefore, the analysis for freeways is based on the travel time estimation for specific scenarios, while the analysis for urban streets identifies analysis scenarios stochastically.

Treatment of Stochasticity in the L08 Framework

The L08 research team decided to use the two different treatments of stochasticity in the L08 freeway and urban street implementations because each approach has advantages and disadvantages and neither is clearly superior to the other in all circumstances. Both implementations of the L08 framework are, in concept, interchangeable; later research by others may reveal which approach is preferred for practical applications.

The primarily deterministic approach implemented in the L08 freeway method assuredly generates all significant probability events, giving the same results each time it is run. However, to keep the computations tractable, it sacrifices explicit consideration of extremely rare events, as well as more frequent events that are judged a priori to be unlikely to significantly affect demand or capacity. Extremely rare, high-impact events are unlikely to effect the overall annual distribution of travel times due to their rarity. They become important when one is concerned about travel times greater than the 95th percentile.

The primarily stochastic approach implemented within the urban streets method does not, a priori, eliminate extremely rare events from consideration. However, given their low probability, they are unlikely to turn up in any given analysis. This strength of the primarily stochastic approach assures the analyst that all possibilities are considered, but the assurance comes at the cost of having to run the analysis several times and average the results. The need for replications ensures that a truly representative range of scenarios is considered. For alternatives analysis, detecting the effects of minor changes to the inputs (such as a modest demand increase or a control change) becomes more difficult because part of the computed difference in travel times may result from stochastic variation. To obtain some confidence in this difference, the analyst must apply a statistical hypothesis test to determine if the observed difference is significant and not the result primarily of chance.

An example of the differences in the two approaches is the generation of incidents for scenarios. The primarily deterministic freeway method considers only three locations and two possible start times for incidents to keep the number of scenarios that have to be modeled to some value significantly below infinity. The primarily stochastic urban streets method considers all possible locations and all possible start times for incidents; but since the method is applied only a deterministic number of times within each scenario, it yields only one of many possible outcomes each time it is applied. The full urban streets analysis must be run several times to obtain the comprehensive power of the stochastic approach to consider incidents in all locations at all times.

Introduction to the Freeway Facilities Methodology

This section provides a high-level description of how travel time reliability can be incorporated in the Freeway Facilities chapter of the HCM2010. The HCM freeway facilities method enables the user to analyze the effect of recurring congestion over an extended facility (about 10 to 15 miles long) and study period (up to 6 hours in duration). This time-space domain allows for the analysis of queue formation and dissipation at bottlenecks, and produces performance measures at the freeway segment, analysis period (15 min), and overall facility levels. Details of the current methodology can be found in Chapter 10 of Volume 2 of the HCM2010 (TRB 2010a) and in Chapter 25 of Volume 4 (TRB 2010b). The computational engine for the methodology, FREEVAL, is also available for download in Volume 4.

The objectives of the L08 project are twofold. The first objective is to incorporate nonrecurring congestion effects into the HCM2010 procedure. The second objective is to expand the analysis horizon from a single study period (typically an a.m. or p.m. peak period) to an extended time horizon of several weeks or months, up to a 1-year reliability reporting period. Together, these objectives lead to a method that allows the analyst to assess the variability in the quality of service that a facility provides to its users. This expanded period, the reliability reporting period, can be thought of as a set of days, each one having its own set of demands and capacities that affect the facility's travel time. This study focused on weather, incidents, work zones, and special events on the supply side, and on volume variability by time of day, day of week, and month of year on the demand side.

Components of the Freeway Facilities Methodology

At its highest level of representation, the freeway facilities methodology has three primary components: a data depository, a scenario generator, and a core computational procedure, which is an adapted and significantly revised version of FREEVAL for reliability, or FREEVAL-RL. These components are illustrated in Figure 4.2.

The largest shaded oval and dotted line represent the current implementation of the freeway facilities method, with study period data specific to the facility being studied entered directly into FREEVAL for analysis of (predominantly) recurring congestion effects. The connection to reliability is enabled by the addition of a scenario generator. Each component and its interaction with the other two are explained in some details in the next sections.

Data Depository

The data depository can be viewed as the virtual space in which all the pertinent data elements needed to execute the methodology reside. Some data are (indeed, some must be) specific to the freeway facility being studied. These data include, at a minimum, all segment geometrics, free-flow speeds, lane patterns, and segment types. Demands can be directly measured for a sample of days from field sensors on the facility, or estimated from projections of annual average daily traffic (AADT) and time-based factors. At a minimum, data must be available to execute one seed file in FREEVAL-RL, much like it is needed to run the current HCM2010 procedure.

Complexities arise when the analyst incorporates sources of nonrecurring congestion effects. Several attributes are required to assess the impact of each source on the facility reliability, including the variations in source type, the probability of its occurrence during the reliability reporting period, and its potential impacts on segment free-flow speed, traffic demand, and segment capacity. An inventory of these attributes is shown in Table 4.1. All data elements are subsequently entered into the scenario generator to start the creation of detailed scenarios to run in FREEVAL-RL and thus estimate travel times.

In summary, the reliability methodology relies on the availability of both facility-specific data elements and default values when data are not available, nonexistent (future analysis), or too expensive to collect. This gives rise to the terms *data-rich* and *data-poor* analyses. However, in most cases, the analysis is a hybrid one, relying on both facility-specific and default data to generate and evaluate scenarios. Thus, each reliability problem could be classified as X% data rich and $1 - X$% data poor, depending on the data availability.

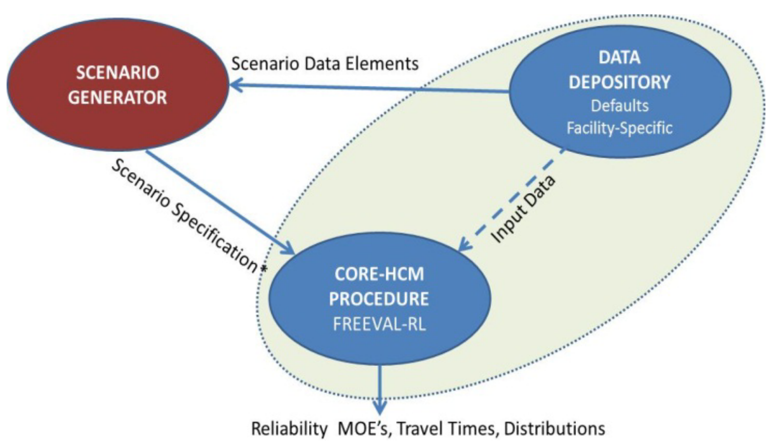


Figure 4.2. Freeway facilities methodology components, including measures of effectiveness (MOEs).

Table 4.1. Inventory of Nonrecurring Congestion Sources and Attributes

Nonrecurring Congestion Source	Elements of Variability	Source for Estimating Probability of Occurrence	Nonrecurring Event Duration	Impact on Segment Free-Flow Speed, Demand, and Capacity
Weather	Nonsevere rain (low, medium, high), snow (light, medium, heavy), visibility (low, minimum), cold, fog	Historical averages by hour and by month[a]; year-specific data	From the same source or national defaults	Extracted from the literature, including HCM2010
Incidents	Shoulder closure; one, two, and three-plus lane closures	Incident logs or rate prediction from crash rates	Incident logs or national defaults	Extracted from the literature, including HCM2010
Work zones	Shoulder closure; one, two, and three-plus lane closures; crossovers	Detailed annual work zone schedules	Detailed traffic control plans for each work zone	Demands must be entered by analyst; capacity effects from literature, including HCM2010
Special events	Shoulder closure; one, two, and three-plus lane closures; crossovers; lane additions; lane reversals	Detailed traffic control plans for each event	Detailed traffic control plans for each event	All free-flow speeds, demands, and capacities must be fully specified by the analyst

[a] For this study, 10-year weather data for 101 metropolitan areas were extracted from Weather Underground (www.wunderground.com).

Scenario Generator

The purpose of the freeway scenario generator (FSG) is to enumerate a sufficiently complete set of operational scenarios that a freeway facility may experience during the reliability reporting period (RRP), along with their associated probabilities. Each scenario represents a single study period that is fully characterized in terms of demand and capacity profiles in time and space. The FSG is flexible, can operate with minimal input (i.e., uses defaults) when data are not available, and accepts facility-specific data when available. All entries are expressed as demand- and capacity-related parameters.

Demand Variability

Demand variations can be entered by time of day, day of week, and month of year (a maximum of 84 demand scenarios for a given study period). The default used in this study is 12 demand scenarios encompassing three weekday types and four seasons. As stated earlier, segment flow rates can be entered directly into a seed file, or estimated on the basis of segment and ramp AADTs in combination with hourly factors, to generate the study period demand. Daily and seasonal demand factors are applied to populate all other scenarios in the reliability reporting period. The only other place that demand patterns may be altered in the scenario generator is in the cases of work zones or special events. Demand in those cases is very much facility- and event-specific and therefore must be directly entered by the user as an input.

Capacity Variability

Much of the focused effort in the FSG is on estimating the probability and impact of nonrecurring congestion, including weather, incidents, work zones, and special events. Data elements are explained in the following sections.

Weather Frequency and Effects

The FSG generates the fraction of RRP time that the facility experiences a particular weather event, along with the impact of the event on capacity and free-flow speed (FFS). The project team extracted durations, for each hour of each month, of 11 HCM-defined weather event types for 101 metropolitan areas in the United States over the most recent 10-year period available. Depending on the application, an analyst can either use data from a specific year or estimate future-year weather probabilities on the basis of long-term historical averages. A screenshot of a weather probability table produced by the FSG is shown in Figure 4.3 along with mean duration and default adjustment factors for capacity and FFS. Each cell gives the fraction of time a weather event is present in the specified month.

When entered in FREEVAL-RL, a weather event is assumed to occur either at the start of the study period or in the middle of the study period, with equal probability, thus generating a maximum of 11 (events) × 2 (start times) or 22 weather scenarios. All the segments on the facility are affected equally by the weather event. When using the FSG to estimate weather probabilities, the analyst simply needs to select the metropolitan area closest to the study facility from a list of 101 national defaults.

Incident Frequency and Effects

Similar to weather, two pieces of information are needed for modeling incidents: (1) the monthly probability of certain incident severities, and (2) the impact of each severity level on capacity. The first piece requires a significant effort to extract

Month	Weather Categories (based on HCM2010 Chapter 10: Freeway Facilities)										
	Med Rain	Heavy Rain	Light Snow	LM Snow	MH Snow	Heavy Snow	Severe Cold	Low Vis	Very Low Vis	Min Vis	Normal Weather
January	1.970%	0.000%	5.911%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	92.1182%
February	2.717%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	2.174%	0.000%	0.000%	95.1087%
March	0.505%	0.000%	1.010%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	98.4848%
April	0.000%	0.543%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	99.4565%
May	1.951%	1.951%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	96.0976%
June	0.505%	0.505%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	98.9899%
July	0.500%	0.500%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	99.0000%
August	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	100.0000%
September	4.255%	0.532%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	95.2128%
October	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	100.0000%
November	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	100.0000%
December	0.000%	0.000%	7.805%	0.488%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	91.7073%
Average Duration for Weather Type(min):	42.9	31.0	134.3	46.6	25.8	5.5	15.0	57.2	15.0	136	
Default Capacity Adjustment Factor:	92.76%	85.87%	95.71%	91.34%	88.96%	77.57%	91.55%	90.33%	88.33%	89.51%	100.00%
Default FFS Adjustment Factor:	93.00%	92.00%	87.00%	86.00%	84.00%	83.00%	93.00%	94.00%	92.00%	92.00%	100.00%

Figure 4.3. FSG-generated weather event probabilities, duration, and impact. Note: LM = light to medium; MH = medium to heavy; Vis = visibility.

the number and duration of each incident from annual incident logs in data-rich environments. Furthermore, the research team's experience with using incident logs revealed significant underreporting of certain incident types. Therefore, a recommended alternative approach is to estimate the facility incident rate from its predicted crash rate, and then use the Poisson process to estimate the likelihood of specific incident severities. Predictive models are available from the Highway Economic Requirements System (HERS) and the *Highway Safety Manual* (HSM) (AASHTO 2010). Capacity adjustments resulting from incidents are taken directly from the HSM2010. A sample incident probability table generated by the FSG is illustrated in Figure 4.4.

Additional incident details must also be generated before running incident scenarios in FREEVAL-RL. These include the following:

- *Incident start time.* Similar to weather, FREEVAL-RL assumes an incident start time either at the start or in the middle of the study period (SP).
- *Incident location.* Three possible locations at the first, middle, and last segment on the facility are included in the FSG.
- *Incident duration.* Based on national averages for incident duration distribution by severity, three representative durations at the 25th, 50th, and 75th percentile values of the distribution are included in the FSG.

Insert Facility Specific	Probability of Different Incident Types					
Month	No Incident	Shoulder Closure	One Lane Closure	Two Lane Closure	Three Lane Closure	Four Lane Closure
January	82.80%	11.96%	3.60%	0.91%	0.73%	0.00%
February	81.57%	12.74%	3.91%	0.99%	0.80%	0.00%
March	81.61%	12.68%	3.91%	1.00%	0.80%	0.00%
April	81.06%	13.06%	4.03%	1.03%	0.82%	0.00%
May	78.50%	14.79%	4.60%	1.18%	0.94%	0.00%
June	81.06%	13.05%	4.03%	1.03%	0.82%	0.00%
July	80.20%	13.64%	4.22%	1.08%	0.86%	0.00%
August	80.49%	13.45%	4.16%	1.06%	0.85%	0.00%
September	81.87%	12.52%	3.85%	0.98%	0.78%	0.00%
October	78.66%	14.67%	4.57%	1.17%	0.93%	0.00%
November	82.51%	12.10%	3.70%	0.94%	0.75%	0.00%
December	84.88%	10.52%	3.16%	0.80%	0.64%	0.00%

Figure 4.4. FSG-generated incident probability matrix.

Thus, a maximum of 2 (start times) × 3 (locations) × 3 (durations) × 5 (severities) = 90 incident scenarios + 1 no-incident scenario = 91 total scenarios. When using the FSG to estimate incident probabilities, the analyst at a minimum needs to provide a facility-specific incident or crash rate.

Work Zone and Special Event Frequency and Effects

Only significant scheduled work zones and special events are considered in the scenario generator. The user must provide the work zone schedule and characteristics (e.g., shoulder work, single-lane closure). In addition, if a significant change in demand is anticipated during the work zone or special event, the appropriate demand values must be entered. Capacity effects of work zones are taken primarily from the existing literature, including the HCM2010. Capacity effects of special events must be entered by the analyst, as those are highly facility- and event-specific.

Generating Scenarios

The FSG assumes that nonrecurring congestion events are independent of each other. Therefore, the probability of an event combination is equal to the product of their two probabilities. The total number of scenarios that will emerge cannot be predicted a priori because only a subset of combinations of demand and capacity variations resulting from the nonrecurring events will occur. However, an upper bound on the number of scenarios can be estimated. Ignoring for the moment the presence of work zones and special events, there are 12 default demand scenarios, 22 weather scenarios, and 91 incident scenarios. If all event combinations have a nonzero probability, then there are approximately 24,000 possible scenarios. In reality, many of the combinations do not exist (e.g., snow in the summer—in most places), and the actual number of generated scenarios is a fraction of the maximum. The generator computes the fractional number of study periods to which each scenario is applicable and divides that number by the reliability reporting period to estimate each scenario's probability. Figure 4.5 shows an example allocation of scenarios, including their descriptions and probabilities. For this real-world facility, 2,508 scenarios are generated, slightly more than 10% of the theoretical maximum.

Core Computation Engine FREEVAL-RL

Interface with the FSG

The FSG will create as many input files for execution in FREEVAL-RL as there are scenarios to analyze. Variations between scenarios result from three types of adjustment factors:

- *Demand variability* by time of day, day of week, and month/season of year is expressed in terms of demand adjustment factors (DAFs) applied to the original demands in the seed file.
- *Capacity variability* resulting from weather, incidents, work zones, and special events is expressed in terms of capacity adjustment factors (CAFs) applied to seed file values; CAFs are applied to specific segments in the cases of incidents or work zones, and facilitywide in the case of weather.
- *Free-flow speed variability* resulting from weather conditions is expressed in terms of free-flow speed adjustment factors (SAFs) applied facilitywide for the duration of the weather event.

Figure 4.6 illustrates a case in which the capacity adjustments for a weather event lasting for 30 min (i.e., two 15-min analysis periods) occurs in combination with an incident on

Detailed Scenario #	Demand Pattern #	WZ and SE	Weather Label	Incident Label	Probability of Detailed Scenario	Incident Event Description			Weather Event Start Time	Number of Incidents	Number of Weather Events	Duration of the Incident (min)	Duration of the Weather Event (min)
						Duration of Incident	Start Time of Incident	Location of Incident					
892	3		Low Vis	Shoulder Closure	0.008998%	Average Duration	Start of SP	Middle Basic Segment	Start of SP	1	1	30	60
893	3		Low Vis	Shoulder Closure	0.008998%	Long Duration	Start of SP	Middle Basic Segment	Start of SP	1	1	45	60
894	3		Low Vis	Shoulder Closure	0.008998%	Short Duration	Start of SP	Last Segment	Start of SP	1	1	30	60
895	3		Low Vis	Shoulder Closure	0.008998%	Average Duration	Start of SP	Last Segment	Start of SP	1	1	30	60
896	3		Low Vis	Shoulder Closure	0.008998%	Long Duration	Start of SP	Last Segment	Start of SP	1	1	45	60
897	2		Normal Weather	Three Lane Closure	0.008530%	Short Duration	Middle of SP	Middle Basic Segment	-	1	0	60	0
898	2		Normal Weather	Three Lane Closure	0.008530%	Average Duration	Middle of SP	Middle Basic Segment	-	1	0	75	0
899	2		Normal Weather	Three Lane Closure	0.008530%	Long Duration	Middle of SP	Middle Basic Segment	-	1	0	75	0
900	2		Normal Weather	Three Lane Closure	0.008530%	Short Duration	Start of SP	Middle Basic Segment	-	1	0	60	0
901	2		Normal Weather	Three Lane Closure	0.008530%	Average Duration	Start of SP	Middle Basic Segment	-	1	0	75	0
902	2		Normal Weather	Three Lane Closure	0.008530%	Long Duration	Start of SP	Middle Basic Segment	-	1	0	75	0
903	3		Med Rain	Shoulder Closure	0.007879%	Short Duration	Middle of SP	First Segment	Middle of SP	1	1	30	45
904	3		Med Rain	Shoulder Closure	0.007879%	Average Duration	Middle of SP	First Segment	Middle of SP	1	1	30	45
905	3		Med Rain	Shoulder Closure	0.007879%	Long Duration	Middle of SP	First Segment	Middle of SP	1	1	45	45
906	3		Med Rain	Shoulder Closure	0.007879%	Short Duration	Middle of SP	Middle Basic Segment	Middle of SP	1	1	30	45
907	3		Med Rain	Shoulder Closure	0.007879%	Average Duration	Middle of SP	Middle Basic Segment	Middle of SP	1	1	30	45
908	3		Med Rain	Shoulder Closure	0.007879%	Long Duration	Middle of SP	Middle Basic Segment	Middle of SP	1	1	45	45
909	3		Med Rain	Shoulder Closure	0.007879%	Short Duration	Middle of SP	Last Segment	Middle of SP	1	1	30	45

Figure 4.5. FSG-generated detailed scenarios and their probabilities.

Segment Number:	1	2	3	4	5	6	7	8	9	10	11	12
Segment Type:	B	ONR	B	OFR	B	W	B	OFR	B	OFR	ONR	B
t=1 (2:00 PM to 2:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=2 (2:15 PM to 2:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=3 (2:30 PM to 2:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=4 (2:45 PM to 3:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=5 (3:00 PM to 3:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=6 (3:15 PM to 3:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=7 (3:30 PM to 3:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=8 (3:45 PM to 4:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=9 (4:00 PM to 4:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=10 (4:15 PM to 4:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=11 (4:30 PM to 4:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=12 (4:45 PM to 5:00 PM)	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.66
t=13 (5:00 PM to 5:15 PM)	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.66
t=14 (5:15 PM to 5:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=15 (5:30 PM to 5:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=16 (5:45 PM to 6:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=17 (6:00 PM to 6:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Figure 4.6. Illustration of CAF application for weather and incidents in FREEVAL input.

the last segment of the facility, also lasting 30 min. The segment CAF reflects the combination of the two events. All adjustment factors by segment and analysis period are sent to the computational engine for processing.

Core Procedural Enhancements

Several enhancements to the original HCM2010 freeway facilities procedure were implemented in the course of the L08 study. These enhancements included (1) developing a method to incorporate FFS and capacity adjustments concurrently, (2) specifying a queue discharge rate less than the uninterrupted-flow capacity, and (3) reporting additional reliability-based outputs in FREEVAL-RL. Each enhancement is discussed in the following sections.

Concurrent SAF and CAF Implementation

To remain in general compliance with the HCM2010, the research team revised the original speed prediction model in HCM2010 Equation 25-1. For basic freeway segments, the new model simply replaces the base free-flow speed with the adjusted free-flow speed, using the appropriate SAF for the prevailing weather conditions. No free-flow speed effects were considered for incidents, as supporting data for this effect were not available in the literature. The revised Equation 25-1 to predict speed (S) for any adjusted flow rate (v_p) assuming a base free-flow speed FFS, base capacity C, and their adjustments SAF and CAF is presented as Equation 4.1.

$$S = (\text{FFS} * \text{SAF}) + \left[1 - e^{\ln\left((\text{FFS} * \text{SAF}) + 1 - \frac{C * \text{CAF}}{45}\right) * \frac{v_p}{C * \text{CAF}}}\right] \qquad (4.1)$$

As a rule, the estimated speed from Equation 4.1 can never exceed the speed at the adjusted capacity (i.e., the speed at a density of 45 passenger cars per mile per lane). This constraint is always satisfied, guaranteeing that the predicted speed will always be at least 1 mph above the estimated speed at capacity.

For ramp and weaving segments, the adjustments to capacity and speed are done independently, since speed estimation for those segment types is independent of capacity. In other words, the CAF is applied to reducing the segment capacity, while the SAF is applied to reducing the FFS and, by extension, the estimated segment speed. The method multiplies c and FFS by CAF and SAF, respectively, throughout the HCM Chapters 12 and 13 methodologies. The implementation of SAF and CAF is detailed in Chapter 6 of this report.

Incorporating Queue Discharge Flow

To more realistically model queue propagation and dissipation on congested freeway facilities, the core procedure now enables the analyst to specify a capacity "loss" resulting from freeway breakdown. This feature was not provided in the HCM2010. However, Hu et al. (2012) found that it has a significant effect on the duration and severity of the congested region. That study found that, according to an extensive literature search, capacity loss averaged 7% during breakdown. In FREEVAL-ML (managed lanes), this value can be entered by the user, although it is restricted to an upper bound of 10% and a minimum of 0% (the current HCM2010 approach). Detailed information on incorporating queue discharge flow in FREEVAL-RL is presented in Chapter 6.

Enhancements to the FREEVAL-RL Outputs

Some of the scenarios run in FREEVAL-RL can clearly generate severe congestion. In some cases, the congestion might be more than the model can handle (e.g., multiple interacting bottlenecks). In addition to providing flags for such occurrences, new performance measures were added to monitor these effects:

- Total number of vehicles denied entry into the facility when the first segment is fully queued; and
- Denied entry vehicles' queue length upstream of segment 1 in each time period.

In addition, the team incorporated new reliability measures to enable comparisons across facilities. For example, (1) the TTI is now calculated and reported for each segment in each analysis period and (2) facility TTI is calculated for each analysis period. Note that each 15-min analysis period contributes one data point to the overall facility travel time distribution.

These measures are part of the standard FREEVAL-RL report, which characterizes the full TTI distribution, along with descriptions of the scenarios that generated the distribution.

Introduction to the Urban Streets Methodology

This section describes the development of a methodology for predicting travel time reliability for urban street facilities. Applying the methodology produces the facility travel time distribution for a specified reliability reporting period. This reliability methodology uses the HCM2010 urban streets methodology to compute facility travel time and other performance measures for each analysis period of interest within the reliability reporting period.

Goals

The research team established several goals to guide the development of a framework for the urban streets reliability methodology. The goals are described in the following list:

- The reliability methodology should use the HCM2010 urban streets methodology to estimate average travel time and other performance measures for a specified analysis period.
- The methodology should quantify the effect of the following sources of nonrecurring congestion: weather events, traffic demand variation, traffic incident occurrence, work zone presence, and special event presence.
- The reliability methodology should minimize the amount of new required input data, beyond that already needed to evaluate an urban street facility for one analysis period (using the HCM2010 urban streets methodology).
- The methodology should provide a default value for each calibration factor used in its component procedures.

Stages

Applying the reliability methodology involves the following three stages, which are implemented in the sequence listed. Each stage is summarized in the subsections below.

- Scenario generation;
- Facility evaluation; and
- Performance summary.

Scenario Generation

In the scenario generation stage, each analysis period in the reliability reporting period is identified. Then, the weather event, traffic demand level, traffic incident occurrence, work zone presence, and special event occurrence are defined for each analysis period. The effect of these factors on segment running speed, intersection saturation flow rate, or signal timing is quantified.

The traffic demand volume, speed, saturation flow rate, and signal timing established for each analysis period are assumed to be unique relative to the other analysis periods. Thus, each analysis period is considered to be one scenario for subsequent evaluation. This assumption recognizes that, in the urban street environment, analysis periods rarely have the same unique combination of demand volume, capacity, and traffic control characteristics for all segments and intersections that make up the facility. The likelihood of unique analysis periods increases when the analysis periods are sequential in a common study period and volumes are sufficiently high that residual queues from one analysis period become initial queues for the subsequent analysis period.

Facility Evaluation

In the facility evaluation stage, each analysis period is evaluated using the computational engine that automates the HCM2010 urban streets methodology. This engine is referred to in this section as the urban streets engine. It is used to estimate the expected value of various performance measures for each intersection and segment, and the facility as a whole, for each analysis period. For any given performance measure, the estimate represents an average for the analysis period and is referred to as an analysis period average (APA).

Performance Summary

In the performance summary stage, the collective set of analysis period results is used to describe a distribution of traffic performance for the reliability reporting period. Facility travel time is the performance measure used to define reliability. However, other performance measures (e.g., intersection delay) can be examined in terms of their variation during the reliability reporting period. Regardless of the performance measure considered, the resulting distribution describes the variation in APA for the reliability reporting period. It does not describe the variation in performance experienced by individual travelers. As a result, some of the variability in performance experienced by individual travelers is not accounted for in this analysis.

Work Flow

The sequence of calculations in the reliability methodology is shown in Figure 4.7. The process is designed around the urban streets engine. It begins with one or more engine input data files. Each file is used to describe the traffic demand, geometry, and signal timing conditions for each intersection and segment on the subject urban street facility for one analysis period.

Most reliability evaluations involve two or more input files. One input file describes base conditions (i.e., when work zones and special events are not present). It is called the base input file. Additional input files are used, as needed, to describe conditions when a specific work zone is present or when a special event occurs. These are called alternative input files.

As a first step in the reliability evaluation, the analyst uses the urban streets engine to generate each of the desired input files. The analyst also identifies the range of dates to which each of the alternative input files is applicable. For example, if an analyst is interested in the travel time during weekday periods from 4:00 to 6:00 p.m. for the current year and the analysis period is 15-min long, then the base input file is used to describe conditions present for one analysis period (say, 4:00 to 4:15 p.m.) when no work zones or special events are present. The demand volumes represent a specified date (provided by the analyst) and can be adjusted in the reliability methodology to estimate volumes for the other dates and times that occur during the reliability reporting period.

If a work zone exists during a given month, then a second input file is used to describe average conditions for the analysis period during that month. As noted previously, the analyst develops this input file using the urban streets engine. The data in the input file reflect the analyst's knowledge of the lane closures and signal timing changes that result from the work zone's presence. The data also reflect the effect of work zone presence on volume, speed, and capacity. The means by which these effects are incorporated in the file is discussed in the Work Zones and Special Events subsection.

Input Data

Once the input files have been created, the data needed to use the reliability methodology are identified. These data are described in Table 4.2.

To identify the typical weather conditions for the subject facility, analysts use the nearest city found in the National Climatic Data Center's (NCDC) publication on climatic data.

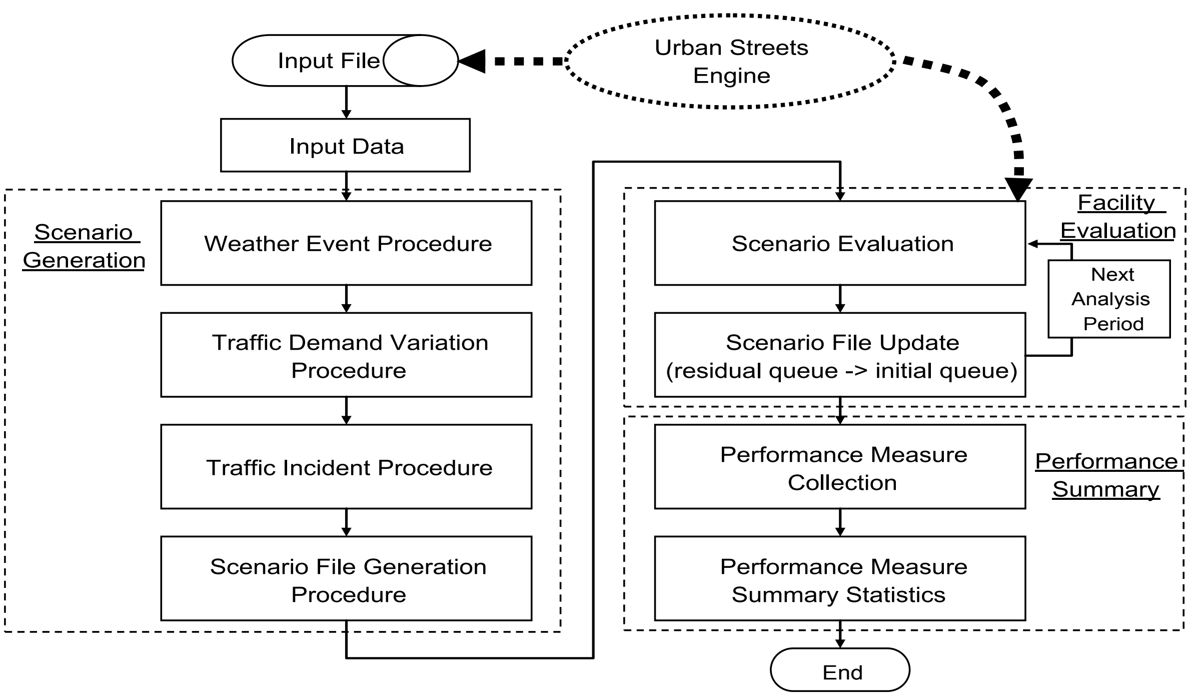


Figure 4.7. Reliability methodology for urban street facilities.

Table 4.2. Input Data

Category	Variable	Description
General	Nearest city	One of 284 U.S. cities and territories whose climatic conditions are summarized periodically by the National Climatic Data Center (www.ncdc.noaa.gov)
	Functional class	Functional class of subject urban street facility
Input file	Date of traffic count	Basis of traffic volumes in base file. Can be either 1. Traffic counts measured in the field (enter the date of the count) or 2. Planning estimates of volume during the average day of week and month of year (do not enter a date)
	Starting hour of the count	Hour of the day that the traffic counts were measured or, if based on planning estimates, hour of the day to which the estimates apply
	Basis of traffic counts in the alternative input files	Basis of traffic volumes in alternative file. Can be either 1. Adjusted traffic counts from base file (enter the date of the count) or 2. Planning estimates of volume when the work zone or special event is present (do not enter a date)
Time period	Analysis period	Duration of analysis period (0.25 h or 1.0 h)
	Study period	Starting hour of study period and its duration in hours
	Reliability reporting period	Starting date of reliability reporting period and its duration in days
	Alternative file operating period	Starting date of work zone or special event and its duration in days
	Days of week considered	Days of week considered in reliability reporting period
Crash	Segment crash frequency	The segment-related crash frequency for each segment, including all severities. The value entered represents the long-run average number of crashes each year when work zones and special events are not present. It is adjusted appropriately if the reliability reporting period is not 1 year in duration.
	Intersection crash frequency	Same as for segments but based on intersection-related crashes
	Crash frequency adjustment factors	This factor is multiplied by the segment or intersection crash frequency. The product represents the long-run crash frequency if the work zone or special event were in operation for 1 year.

The Center periodically publishes summaries from weather stations in each of 284 U.S. cities and territories (NCDC 2011a). The document contains 17 statistics related to temperature, wind, cloudiness, humidity, and precipitation. Each statistic is quantified by month of year and based on 10 or more years of data. Of interest to reliability evaluation are the following weather statistics from this document:

- Total normal precipitation;
- Total normal snowfall;
- Number of days with precipitation of 0.01 in. or more; and
- Normal daily mean temperature.

The NCDC also provides storm event data for several thousand locations throughout the United States (including the aforementioned 284 locations). These data describe the average number of storms, average precipitation depth per storm, average storm duration, and average precipitation rate (i.e., intensity). Each statistic is quantified by month of year. Of interest to reliability evaluation is the average precipitation rate in the *Rainfall Frequency Atlas* (NCDC 2011b).

The functional class of the subject facility is used to determine the appropriate month-of-year and hour-of-day traffic volume adjustment factors. Hallenbeck et al. (1997) examined continuous count station data from 19 states and found that these factors varied by functional class. They also noted some difference in factor values when comparing the coastal states with the Great Plains and Rocky Mountains. Their report was used as the basis for the default month-of-year, hour-of-day, and day-of-week adjustment factors described in Appendix H. The latter set of factors was not found to be sensitive to functional class, but the factors were sensitive to area type (i.e., urban or rural).

The starting hour of the count is used to determine the hour-of-day adjustment factor. This factor, with the month-of-year and day-of-week factors, is then used to convert the volumes in the base input file into average-day-of-year volumes. A similar adjustment is made to the volumes in the alternative input files. During the scenario file generation, these averages are used to estimate the volume for specific hours and days of the year.

The crash frequency data are used to estimate the frequency of non-crash-related incidents. The procedure for computing this estimate is described in a subsequent section. For urban streets evaluation, crashes are categorized as

- Crashes related to the segment; or
- Crashes related to the intersection.

The two categories are mutually exclusive. A technique for determining whether a crash is a segment- or intersection-related crash is described in Appendix A to Part C of the HSM (AASHTO 2010). The crash frequency that is input represents an estimate of the expected crash frequency for base traffic demand volume, geometry, and signal timing conditions. The estimate should include all severity levels, including property-damage-only (PDO) crashes. It is provided in units of crashes per year, regardless of the duration of the reliability reporting period. The procedure uses the expected crash frequency to estimate the number of crashes that occur during the reliability reporting period.

The expected crash frequency can be computed by using the predictive method in Chapter 12 of the HSM. If this method cannot be used, then a 3-year crash history for the subject facility can be used to estimate the expected crash frequency. Crashes that occur when work zones and special events are present should be removed from the crash data. In this situation, the expected crash frequency is computed as the count of crashes during base conditions divided by the time period (in years) when base conditions are present.

The crash frequency adjustment factor is used to estimate the expected crash frequency when a work zone or special event is present. This factor is multiplied by the expected crash frequency for base conditions. The product represents the expected crash frequency if the work zone or special event was in operation for 1 year.

The factor value should include consideration of the effect of the work zone or special event on traffic volume (i.e., volume may be reduced because of diversion) and on crash risk (i.e., the geometry and signal operation changes for the work zone or special event may increase the potential for a crash). For example, if a work zone is envisioned to increase crash risk by 100% (i.e., crash risk is doubled) and to decrease traffic volume by 50% (i.e., volume is halved), then the crash frequency adjustment factor is 1.0 (= 2.0 × 0.5). The analyst's experience with similar types of work zones or special events should be used to determine the appropriate adjustment factor value for the subject facility.

Scenario Generation

The scenario generation stage consists of four sequential procedures which are described in more detail in Chapter 5. Each procedure processes the set of analysis periods in chronologic order.

The first procedure predicts weather event date, time, type (i.e., rain or snow), and duration. The second procedure identifies the appropriate traffic volume adjustment factors for each date and time during the reliability reporting period. These factors are used during the scenario file generation procedure to estimate the volume associated with each analysis period. The third procedure predicts incident event date, time, and duration. It also determines incident event type (i.e., crash or noncrash), severity level, and location on the facility. It uses weather event and demand variation information from the two previous procedures in the incident prediction process.

The fourth procedure uses the results from the preceding three procedures to develop one urban streets engine input file for each analysis period in the reliability reporting period. As discussed previously, each analysis period is considered to be one scenario. Date and time represent a common basis for linking the events and conditions related to all four procedures. Each input file created in this procedure includes the appropriate adjustments to segment running speed and intersection saturation flow rate associated with the weather or incident events that occur during the corresponding analysis period. Similarly, the traffic demand volumes in each file are adjusted for monthly, weekly, and hourly variations.

Variance Control

Weather events; traffic demand; and traffic incident occurrence, type, and location have both systematic and random elements. To the extent practical, the reliability methodology accounts for the systematic variation component in its predictive models. Specifically, it recognizes changes in weather and traffic demand depending on time during the year, month, and day. It also recognizes the influence of geographic location on weather and the influence of weather and traffic demand on incident occurrence.

Models of the systematic influences are included in the methodology. They are used to predict average weather, demand, and incident conditions during each analysis period. However, the use of averages to describe weather events and incident occurrence for such short time periods is counter to the objectives of reliability evaluation. The random element of weather events, demand variation, and traffic incident occurrence introduces a high degree of variability in the collective set of analysis periods that make up the reliability reporting period. Thus, it is important to replicate these random elements in any reliability evaluation. Monte Carlo methods are used for this purpose in the urban streets reliability method.

A random number seed is used with the Monte Carlo methods in the reliability methodology so that the sequence of random events can be reproduced. In fact, a unique seed number is separately established for weather events, demand variation, and incident occurrence. For a given set of three seed numbers, a unique combination of weather events, demand levels, and incidents is estimated for each analysis period in the reliability reporting period.

One, two, or three of the seed numbers can be changed to generate a different set of conditions, if desired. For example, if the seed number for weather events is changed, then a new series of weather events is created and, to the extent that weather influences incident occurrence, a new series of incidents is created. Similarly, the seed number for demand variation can be used to control whether a new series of demand levels is created. The seed number for incidents can be used to control whether a new series of incidents is created.

When evaluating alternatives, analysts will likely use one set of seed numbers as a variance reduction technique. In this application, the same seed numbers are used for all evaluations. With this approach, the results from an evaluation of one alterative can be compared with those from an evaluation of the baseline condition. Any observed difference in the results can be attributed to the changes associated with the alternative (i.e., they do not result from random changes in weather or incident events among the evaluations).

Replications

A complete exploration of reliability would likely entail the use of multiple, separate evaluations of the same reliability reporting period with each evaluation using a separate set of random number seeds. This approach may be particularly useful when the facility has infrequent severe weather events or incidents. With this approach, the evaluation is replicated multiple times and the performance measures from each replication are averaged to produce a more reliable estimate of their long-run value.

Facility Evaluation

The facility evaluation stage consists of two tasks that are repeated in sequence for each analysis period. The analysis periods are evaluated in chronologic order.

For the first task, the input file associated with an analysis period is submitted to the urban streets engine for evaluation. Then, the predicted performance measures for the subject analysis period are saved to an output file with a unique file name.

During the second task, the performance measures are extracted from the output file and used to revise the input file associated with the next analysis period. Specifically, the input file for the next analysis period is read, modified, and saved before returning to the first task. The modification entails setting the initial queue input value for the next analysis period equal to the residual queue output from the current analysis period.

Sampling Technique

Typical combinations of reliability reporting period, analysis period, weather event occurrence, and incident event occurrence often produce a large number of scenarios. The collective evaluation of these scenarios could take an hour or more when the methodology is automated in software. This length of time may be considered too long for some reliability applications, in which case a sampling approach is available.

A sampling technique can be used to minimize the total evaluation time. The analyst needs to input the scenario evaluation interval. The interval has units of days. The analyst can choose to evaluate every scenario for every day (i.e., input "1"). Alternatively, the analyst can chose to evaluate every scenario for every other day (i.e., input "2"). More generally, the analyst can input any integer number for the evaluation interval.

The evaluation interval is checked to ensure that all days in the reliability reporting period are equally sampled. The check examines the pattern produced by the input "days of week considered" D and the evaluation interval I. An interval factor F is computed as $F = I - \text{int}[I/D] \times D$. If 5 or 7 days of the week are considered, then values of I that yield $F > 0$ provide the desired representative sample. If 2, 3, 4, or 6 days of the week are considered, then values of I that yield $F = 1$ or $F = D - 1$ provide the desired sample.

Performance Summary

The performance summary stage consists of two sequential tasks. The first task reads the output file for each analysis period and collects the desired performance measure. At the start of this task, the analyst identifies the specific direction of travel and the performance measure of interest, selecting from the following list:

- Travel time;
- Travel speed;
- Stop rate;
- Running time; and
- Through delay.

The analyst also indicates whether the performance measure of interest represents the entire facility or a specific segment. The first three measures in the list are available for facility evaluation. All five measures are available for segment evaluation. At the conclusion of this task, the collected data represent observations of the performance measure for the each analysis period occurring during the reliability reporting period (or a sampled subset).

During the second task, the selected performance measure data are summarized using the following statistics:

- Average;
- Standard deviation;
- Skewness;
- Median;

- 10th, 80th, 85th, and 95th percentiles; and
- Number of observations.

In addition, the average base free-flow speed is always reported. It can be used with one or more of the distribution statistics to compute various reliability measures, such as the TTI.

Work Zones and Special Events

Work zones and special events influence traffic demand levels and travel patterns. To minimize the impact of work zones and special events on traffic operation, agencies responsible for traffic accommodation in the vicinity of the work zone or special event often reallocate some traffic lanes or alter the signal operation to increase the capacity of specific traffic movements. These characteristics make each work zone and special event unique, and their effect on facility performance equally unique. Different work zones and special events can occur during the reliability reporting period.

The reliability methodology incorporates work zone and special event influences in the evaluation results. However, the analyst must describe each work zone and special event using an alternative input file. Each file describes the traffic demand, geometry, and signal timing conditions when the work zone is present or the special event is under way. A start date and duration are associated with each file.

Work zone presence can have a significant effect on traffic demand levels. The extent of the effect depends partly on the availability of alternate routes, the number of days the work zone has been in operation, and the volume-to-capacity ratio of the segment or intersection approach with the work zone. Lee and Noyce (2007) evaluated motorist response to several freeway work zones in Wisconsin. They concluded that diversion resulted in a volume reduction of 40% to 50%. This conclusion was based on their comparison of the observed queue forming upstream of the work zone with that predicted using volumes measured during normal-day operations.

When using the reliability methodology, the analyst must provide an estimate of traffic demand volumes during the work zone or special event. These estimates should reflect the effect of diversion, and they can be based on field measurements, judgment, or area-wide traffic planning models. They are recorded by the analyst in the corresponding alternative input file.

The analyst must have information about lane closures, alternative lane assignments, and special signal timing that are present during the work zone or special event. This information can be based on agency policy or on experience with previous work zones or events. The available lanes, lane assignments, and signal timing are recorded by the analyst in the corresponding alternative input file.

A review of the literature indicates that work zone presence can affect intersection saturation flow rate. An adjustment factor for this effect is described in Appendix I. It can be used with the saturation flow rate prediction procedure in Chapter 18 of HCM2010 to estimate the saturation flow rate when a work zone is present. The analyst then enters the adjusted saturation flow rate in the appropriate alternative input file. These adjustments are not made as part of the reliability methodology.

CHAPTER 5

Scenario Generator Development

This chapter discusses the development of scenario generators for freeway facilities and urban streets. It is divided into the following seven sections:

1. Introduction to freeway scenario development;
2. Concept and generation of base freeway scenarios;
3. Study period for freeway scenario generation;
4. Detailed freeway scenario generation;
5. Freeway scenario generation input for FREEVAL-RL;
6. Freeway summary and conclusions; and
7. Urban street scenario development.

Introduction to Freeway Scenario Development

The freeway scenario generator (FSG) generates and assigns initial probabilities to a number of base scenarios, Each base scenario is a combination of events that occur within a given time period, typically a weekday or (more likely) a few hours thereof. A base scenario probability is expressed as the fraction of time a particular combination of events takes place during the study period (SP) of interest (e.g., the a.m. or p.m. peak period). In this project, a scenario is a specific, unique realization of the study period, which may or may not contain a combination of weather and/or incident events. Base scenario probabilities are computed assuming independence between the events and at the initial stage and do not take into account the actual event duration. The base scenarios only account for the categories of weather and/or incidents. Therefore, the initial probabilities must be adjusted to account for the actual event duration and, in some cases, the scenario definition requires detailed adjustment of the event durations. This adjustment process is extensive and complex.

The FSG is a deterministic approach to scenario generation. This deterministic approach enumerates different operating conditions of a freeway facility on the basis of different combinations of factors which affect travel time. The distinct sets of operational conditions are expressed as operational scenarios or, simply, scenarios. Four principal steps explain the construction of the scenario generation process for freeway facility analysis, as depicted in Figure 5.1.

The FSG can work both in data-rich and data-poor environments, as well as in data environments that lie between the two extremes. In the data-rich case, the user is asked to input as much local data as possible. When local data are unavailable, the FSG relies on national defaults to generate the scenarios. At a minimum, the user must enter information regarding the subject facility seed file demand, geographic location, and detailed geometrics. The minimum data requirements are similar to the data requirements for most current HCM analysis procedures.

Demand is entered into the FREEVAL-RL seed file. Detailed data such as daily and monthly demand variations are also needed. The FSG allows the user to enter facility-specific demand data or to use national default values for demand pattern definitions. The FSG also provides 10-year average weather data for 101 metropolitan areas (based on weather data from 99 airports), which users can apply in the absence of site-specific weather data. In addition, the FSG provides a flexible procedure for incident data entry that enables the analyst to use as much or as little facility-specific data as is appropriate for characterizing the probability of various incident types. More detailed information on incident probability is available in Appendix F.

Basic Definitions

Analysis period (AP) is the 15-min time interval for which segment and facility operations are calculated in the HCM2010 freeway facility methodology.

Base scenarios enumerate the mutually exclusive *states* or *combinations* of demand, weather, incident, work zone, and special event categories that occur on a freeway facility. The d/c probability of each scenario indicates the portion

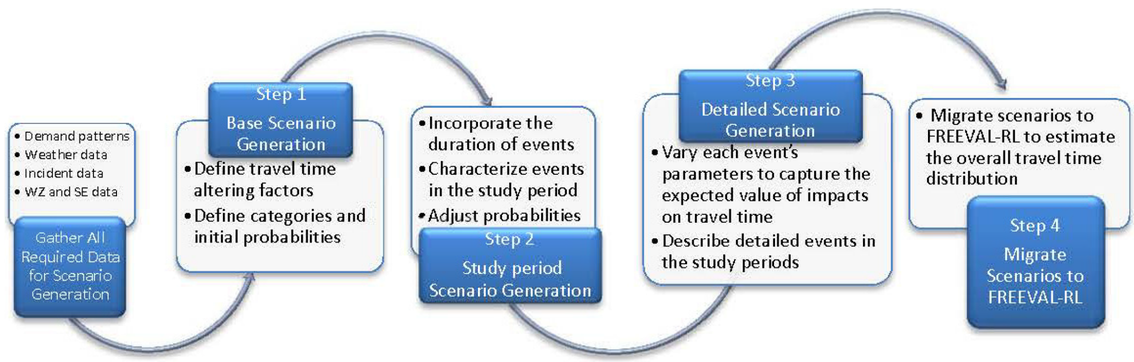


Figure 5.1. Process flow overview for freeway scenario generation. Note: WZ = work zone; SE = special event.

of time in the reliability reporting period (RRP) that the facility is expected to be operating under this condition.

Demand pattern represents a combination of days within the RRP that have similar daily and monthly demand levels.

Detailed scenarios are realizations or details of some SP scenarios using varying incident or weather event attributes. These scenarios implement the variability in the event start time, duration, and location. For example, each defined weather event is modeled twice: once when it occurs at the start of a SP, and once when it occurs in the middle. Similarly, for each base incident scenario, the duration, start time, and location of the incident is allowed to vary across the related detailed scenarios.

Event is any incident or severe weather occurrence expected to affect facility travel time.

FREEVAL-RL is a version of the HCM2010 computational engine for freeway facilities that has been enhanced for reliability analysis.

Freeway scenario generator (FSG) is a methodology to generate detailed scenarios that can capture the effects of recurring and nonrecurring congestion on travel time variability. The generator is implemented in Visual Basic for Applications (VBA)/Excel module with input and output worksheets, which enables the user to generate detailed scenarios to be executed in the computational engine for freeway reliability, FREEVAL-RL.

Normal condition is the condition without severe weather or incidents (i.e., the absence of weather or incident events producing more than a negligible impact on demand and capacity).

Parent scenario is a scenario that serves an *identical* demand pattern to that used in a particular detailed scenario containing weather and/or incident events. Each detailed scenario has a parent scenario. Defining parent scenarios enables the analyst to assess the incremental effect of incidents and weather events on facility travel time by analyzing the *differences* in travel times between a detailed scenario and its parent scenario.

Reliability reporting period (RRP) is an extended time horizon, typically a year, over which the analysis is carried out.

Study period (SP) is the time frame within a single day over which freeway facility reliability is analyzed. It usually covers the a.m. or p.m. peak period. The study period is the sum of the sequential analysis periods for which the HCM2010 facility analysis procedure is applied (e.g., a 4-hour peak period).

Study period scenarios are combinations of base scenarios that describe what occurs during the course of a study period. They specify events and their duration inside study periods. The SP scenarios represent the expected conditions under which the subject freeway facility will operate during the study period.

Concept and Generation of Base Freeway Scenarios

Demand level, weather, and incidents are the three contributors to freeway facility travel time variability considered in the FSG. These factors introduce stochasticity to travel time. In other words, considering and modeling a statistically representative set of scenarios that includes these factors across the RRP generates a travel time distribution for the RRP.

The freeway scenario generation process uses a deterministic approach to model these variations. It categorizes different sources of variability (e.g., demand patterns, incident types) into different subcategories. For instance, weather, which is one of the main contributors to travel time variability, is defined in 11 weather categories such as nonsevere weather, medium rain, or snow. Each category has a time-wise probability of occurrence and an impact on facility capacity and speed.

Base Scenario Assumptions

Contributing factors to travel time variability are assumed to be independent. The FSG methodology does allow some factors, such as demand, to vary by other factors, such as weather type. However, explicit consideration of factor interactions of this type must be handled during postprocessing of the automatically generated detailed scenarios.

The contributing factors to travel time variability are sorted into discrete categories with a time-wise probability of occurrence, which are neither frequencies nor chances of occurrence. If time-wise probabilities of occurrences are not available, appropriate methodologies are described in Appendices E and F to help estimate the probabilities.

The time unit for scenario generation is minutes. Every calculation for measuring the probabilities is based on minutes.

Another simplifying assumption in the FSG is that any time instance within the SP and across the RRP is independent of other time instances. For example, the condition on the freeway at 1:45 p.m. on January 12, 2012, is independent of the conditions in any other 1-min period in the RRP or SP—such as 1:44 p.m. on January 12, 2012, or 3:25 p.m. on March 21, 2012.

Required Input Data for Generating Base Scenarios

To calculate the base scenario probabilities, the time-wise probabilities of different types of contributors to the variation in the travel time distribution should be known. The variation in these factors should be allocated to certain categories, with associated probabilities. The incident and weather probabilities do not deal with the frequency or counts of those events. However, event frequencies can be estimated on the basis of the time-wise probabilities and the expected duration of the different event types.

Demand Variability

Categorization of demand is done by defining demand patterns (DPs) in the RRP. Specific days with similar demand levels are assigned to one demand pattern. The basis of defining a demand pattern consists of two dimensions, which account for the monthly and weekly variability of demand in the RRP. Monthly variability usually highlights seasonal demand effects, while the weekly dimension shows the effect of daily variations in demand levels.

The demand level should be studied for the facility where the reliability analysis is performed. As one of the requirements, demand multipliers (DMs) should be compiled for each day for all months in the RRP. The demand multipliers give the ratio of demand for a day-month combination to the AADT and are used to generate demand values for later FREEVAL-RL runs. More detailed discussion is provided in the Detailed Freeway Scenario Generation section of this chapter. In the absence of facility-specific demand multipliers, the FSG defaults to embedded urban or rural default values. Table 5.1 shows the demand multipliers for the I-40 eastbound (EB) case study. Explanation of the colors in the table follows.

Demand patterns are defined according to the demand multiplier distribution across the various study months and days. This task is performed by the analyst, although the user can select the FSG default demand pattern. For example, the

Table 5.1. Demand Multipliers for I-40 EB Case Study

	Day of Week				
Month	**Monday**	**Tuesday**	**Wednesday**	**Thursday**	**Friday**
January	0.996623	1.027775	1.040394	1.052601	1.081612
February	0.939253	1.010728	1.039214	1.092029	1.140072
March	1.043305	1.069335	1.063524	1.110921	1.171121
April	1.073578	1.087455	1.098238	1.161974	1.215002
May	1.076331	1.106182	1.113955	1.157717	1.210434
June	1.078043	1.085853	1.067470	1.138720	1.180327
July	1.082580	1.070993	1.102512	1.147279	1.184981
August	1.046045	1.052146	1.060371	1.093243	1.164901
September	1.016023	1.024051	1.023625	1.074782	1.152946
October	1.048981	1.045723	1.066986	1.107044	1.160954
November	0.974044	0.999947	1.041211	1.081541	1.070354
December	0.974785	0.956475	0.987019	0.916107	1.007695

Table 5.2. Demand Pattern Configuration for I-40 EB Case Study

	Monday	Tuesday	Wednesday	Thursday	Friday
January	1	1	1	2	3
February	1	1	1	2	3
March	4	4	4	5	6
April	4	4	4	5	6
May	4	4	4	5	6
June	7	7	7	8	9
July	7	7	7	8	9
August	7	7	7	8	9
September	10	11	11	12	12
October	10	11	11	12	12
November	10	11	11	12	12
December	1	1	1	2	3

demand pattern for the I-40 EB case study was found to be seasonal across the monthly dimension. Furthermore, demand on Mondays, Tuesdays, and Wednesdays could be considered as one group, while Thursdays and Fridays were unique and classified as two additional, separate groups. The demand pattern definition for I-40 EB (Table 5.2) is based on comparing demand levels and categorizing days of the week and months of the year according to the demand level shown in Table 5.1. The text color entries in Table 5.1 reflect the same collection of patterns.

To estimate the probability of each demand pattern, the fraction of the RRP (in minutes) with a certain demand pattern is divided by the total RRP duration. Table 5.3 presents a schematic of FSG demand patterns associated with the I-40 EB case study. The demand pattern number (shown in parentheses following the date) provides a simple indicator of each day's demand level. The FSG begins with the first calendar day of the RRP and assigns a demand pattern number to each day within the RRP.

The probability of demand pattern Z, expressed as $p_D(Z)$, is computed by using Equation 5.1.

$$p_{\mathrm{DP}}(Z) = \frac{\text{Sum of SP minutes within demand pattern } Z}{\text{Sum of SP minutes in RRP}} \tag{5.1}$$

For example, the probability of occurrence of demand pattern 5 at any time in the RRP is shown below:

$$p_{\mathrm{DP}}(5) = \frac{13 \times 6 \times 60}{261 \times 6 \times 60} = 4.98\%$$

where the number of SPs (or days) with demand pattern 5 is 13, SP is equal to 6 hours, and the total number of SPs in the RRP (or days in analysis) is 261.

Weather Variability

In the HCM2010, weather events are divided into 16 categories (including normal). Five categories have a negligible effect on the performance of the freeway facility and travel time. The remaining 11 categories are considered in this methodology. The probabilities of these 11 categories are stated by month, which enables the analyst to incorporate the effect of seasonal changes in the weather into the reliability analysis. A detailed discussion about the generation of nationwide weather categories for freeway reliability analysis can be found in Appendix E.

In data-rich environments—in which analysts have access to detailed local weather data—the probability of a weather category is computed using Equation 5.2. Weather categories are mutually exclusive, so when two or more categories can be identified for the same time period (e.g., low visibility and heavy rain), the event is assigned to the category with largest capacity reduction effect in Equation 5.2. Weather in each category is called *weather type.*

$$p_w(i, j) = \frac{\text{Sum of all SP durations in minutes in month } j \text{ that weather type } (i) \text{ is present}}{\text{Sum of all SP durations in minutes in month } j} \tag{5.2}$$

Where $p_w(i, j)$ is the probability of encountering weather type i in month j. In the absence of local data, the FSG

Table 5.3. Partial Listing of Demand Patterns Associated with I-40 EB Case Study

Week No.	Month	Monday	Tuesday	Wednesday	Thursday	Friday
1	January	N/A	N/A	N/A	N/A	1/1/2010 (3)
2	January	1/4/2010 (1)	1/5/2010 (1)	1/6/2010 (1)	1/7/2010 (2)	1/8/2010 (3)
3	January	1/11/2010 (1)	1/12/2010 (1)	1/13/2010 (1)	1/14/2010 (2)	1/15/2010 (3)
4	January	1/18/2010 (1)	1/19/2010 (1)	1/20/2010 (1)	1/21/2010 (2)	1/22/2010 (3)
5	January	1/25/2010 (1)	1/26/2010 (1)	1/27/2010 (1)	1/28/2010 (2)	1/29/2010 (3)
6	February	2/1/2010 (1)	2/2/2010 (1)	2/3/2010 (1)	2/4/2010 (2)	2/5/2010 (3)
7	February	2/8/2010 (1)	2/9/2010 (1)	2/10/2010 (1)	2/11/2010 (2)	2/12/2010 (3)
8	February	2/15/2010 (1)	2/16/2010 (1)	2/17/2010 (1)	2/18/2010 (2)	2/19/2010 (3)
9	February	2/22/2010 (1)	2/23/2010 (1)	2/24/2010 (1)	2/25/2010 (2)	2/26/2010 (3)
10	March	3/1/2010 (4)	3/2/2010 (4)	3/3/2010 (4)	3/4/2010 (5)	3/5/2010 (6)
11	March	3/8/2010 (4)	3/9/2010 (4)	3/10/2010 (4)	3/11/2010 (5)	3/12/2010 (6)
12	March	3/15/2010 (4)	3/16/2010 (4)	3/17/2010 (4)	3/18/2010 (5)	3/19/2010 (6)
13	March	3/22/2010 (4)	3/23/2010 (4)	3/24/2010 (4)	3/25/2010 (5)	3/26/2010 (6)
14	April	3/29/2010 (4)	3/30/2010 (4)	3/31/2010 (4)	4/1/2010 (5)	4/2/2010 (6)
15	April	4/5/2010 (4)	4/6/2010 (4)	4/7/2010 (4)	4/8/2010 (5)	4/9/2010 (6)
16	April	4/12/2010 (4)	4/13/2010 (4)	4/14/2010 (4)	4/15/2010 (5)	4/16/2010 (6)
17	April	4/19/2010 (4)	4/20/2010 (4)	4/21/2010 (4)	4/22/2010 (5)	4/23/2010 (6)
18	May	4/26/2010 (4)	4/27/2010 (4)	4/28/2010 (4)	4/29/2010 (5)	4/30/2010 (6)
19	May	5/3/2010 (4)	5/4/2010 (4)	5/5/2010 (4)	5/6/2010 (5)	5/7/2010 (6)
20	May	5/10/2010 (4)	5/11/2010 (4)	5/12/2010 (4)	5/13/2010 (5)	5/14/2010 (6)
21	May	5/17/2010 (4)	5/18/2010 (4)	5/19/2010 (4)	5/20/2010 (5)	5/21/2010 (6)
22	May	5/24/2010 (4)	5/25/2010 (4)	5/26/2010 (4)	5/27/2010 (5)	5/28/2010 (6)
23	June	5/31/2010 (4)	6/1/2010 (7)	6/2/2010 (7)	6/3/2010 (8)	6/4/2010 (9)
24	June	6/7/2010 (7)	6/8/2010 (7)	6/9/2010 (7)	6/10/2010 (8)	6/11/2010 (9)

Note: N/A = not applicable.

provides a VBA module and weather database for computing the probability of different weather types as a function of the facility's geographic location, and time of day coincident with the SP. The weather database includes 10-year averages of hourly logs for 101 metropolitan areas in the United States. Table 5.4 presents the weather probabilities for the I-40 EB case study in 2010. For example, the probability of a medium rain event between 2:00 and 8:00 p.m. in May 2010 is shown to be p_w (Med Rain, 5) = 1.951%.

When using the 10-year average weather probabilities, a threshold is set in the FSG to remove weather events with very low probabilities, thus reducing the overall number of scenarios. The threshold is specified by the user. Any weather event with a probability lower than the threshold is removed, and its probability is assigned to the remaining weather events proportionally on the basis of their probabilities. The default value for this threshold is 0.1%. Entering a value of zero for the threshold disables its functionality. It is not recommended to enter a large value for this threshold, because doing so could result in a significant loss of fidelity in the estimated travel time distribution.

Incident Variability

Incidents are categorized according to their severity or capacity impacts. For the purpose of scenario generation, six categories are defined for characterizing the incident effect. Because of the complexity of estimating the probability of incidents on the freeway facility, the FSG provides multiple options for analysts to use the available incident or crash data to generate the monthly incident probabilities. The resolution of incident probabilities is months. The

Table 5.4. Weather Probabilities for I-40 EB Case Study

	Weather Categories (based on HCM2010 Chapter 10: Freeway Facilities)										
Month	Medium Rain (%)	Heavy Rain (%)	Light Snow (%)	Light to Medium Snow (%)	Medium to Heavy Snow (%)	Heavy Snow (%)	Severe Cold (%)	Low Visibility (%)	Very Low Visibility (%)	Minimal Visibility (%)	Normal Weather (%)
January	1.970	0.000	5.911	0.000	0.000	0.000	0.000	0.000	0.000	0.000	92.1182
February	2.717	0.000	0.000	0.000	0.000	0.000	0.000	2.174	0.000	0.000	95.1087
March	0.505	0.000	1.010	0.000	0.000	0.000	0.000	0.000	0.000	0.000	98.4848
April	0.000	0.543	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	99.4565
May	**1.951**	1.951	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	96.0976
June	0.505	0.505	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	98.9899
July	0.500	0.500	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	99.0000
August	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	100.0000
September	4.255	0.532	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	95.2128
October	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	100.0000
November	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	100.0000
December	0.000	0.000	7.805	0.488	0.000	0.000	0.000	0.000	0.000	0.000	91.7073

probability of incident type i in month j is computed from Equation 5.3:

$$p_{\text{Inc}}(i,j)=\frac{\text{Sum of all SP durations in minutes in month } j \text{ that incident type } (i) \text{ is present}}{\text{Sum of all SP durations in minutes in month } j} \tag{5.3}$$

If local incident probabilities are not available for a facility, then using either local crash rates or crash rates predicted from the HERS model in combination with an incident-to-crash ratio enables one to calculate the probabilities of different incident types. A queuing model is used for computing the probability of having incidents in the freeway facilities. A more detailed discussion of incident generation is provided in Appendix F.

Table 5.5 depicts the probabilities associated with incidents for the I-40 EB case study. For example, the probability of an incident causing a single-lane closure anywhere on the facility between 2:00 and 8:00 p.m. in May 2010 is p_{Inc} (One-Lane Closure, 5) = 7.64%.

Independence of Time Instances (Minutes) and Joint Events

The stated probabilities of events are associated with and connected to the frequency of event occurrence. However, the FSG computes the time-wise probabilities of encountering different categories of contributors to variations in travel time. Specifically, the probability of each subcategory yields the chance of exposure to a specified category at any instance in the RRP or SP. From a mathematical perspective, the duration of the weather or incident events are not considered at the base scenario generation stage. Any time instance in the RRP or SP is therefore assumed to be independent of any other time instance. More precisely, if the state of any contributor to travel time variation at any time instance is known, the methodology assumes that this state has no effect on the probability of encountering any other contributing factor in the remaining time instances. The units for measuring the probabilities of subcategories are minutes. Therefore, the time instance refers to any 1-min time interval in the SP or RRP.

This basic assumption that all contributing factors to travel time variation are independent allows one calculate the probability of a base scenario as the product of the probability of all contributing factors. For example, given the assumption that there is no dependency between certain demand levels and different weather types, the methodology combines these categories and multiplies their probabilities to generate the different operational conditions and associated probabilities for the freeway facility. These scenarios are referred to as base scenarios.

Equation 5.4 is used to calculate the joint probability of each base scenario based on the scenario's probability of weather and incident events, assuming independence between factors.

Table 5.5. Incident Probabilities for I-40 EB Case Study

Month	Probability of Different Incident Types					
	No Incident (%)	Shoulder Closure (%)	One-Lane Closure (%)	Two-Lane Closure (%)	Three-Lane Closure (%)	Four-Lane Closure (%)
January	66.42	23.30	7.06	1.79	1.43	0.00
February	66.36	23.34	7.08	1.79	1.43	0.00
March	65.10	24.18	7.36	1.87	1.49	0.00
April	63.79	25.05	7.66	1.94	1.56	0.00
May	63.87	25.00	**7.64**	1.94	1.55	0.00
June	64.53	24.56	7.49	1.90	1.52	0.00
July	64.10	24.85	7.59	1.93	1.54	0.00
August	65.30	24.04	7.32	1.86	1.48	0.00
September	65.97	23.60	7.17	1.82	1.45	0.00
October	65.04	24.22	7.38	1.87	1.50	0.00
November	66.79	23.05	6.98	1.77	1.41	0.00
December	68.56	21.86	6.59	1.67	1.33	0.00

$$\begin{aligned}&\text{Prob}\{\text{Demand Level } i, \text{Weather Type } j, \text{Incident Type } k\}\\&\quad = \text{Prob}\{\text{Demand Level } i\} \times \text{Prob}\{\text{Weather Type } j\}\\&\qquad \times \text{Prob}\{\text{Incident Type } k\}\end{aligned} \tag{5.4}$$

Note that some dependencies between event occurrences are inherent through the use of the calendar. It is intuitively obvious and observable from data that both demand levels (in Table 5.2) and weather conditions (in Table 5.4) are associated with the calendar. Therefore, a correlational (not a causal) relationship exists between the two factors. Incident probabilities are also tied to the prevailing demand levels, again providing a correlation through the calendar. In fact, the user can enter different monthly crash or incident rates in the FSG to express further associated weather and incident probabilities.

Aggregation of Probabilities Across Demand Patterns

Each base scenario is characterized by a demand pattern, weather event, and incident type. Given this characterization, the probability of each scenario can be computed. However, the probability of weather and incidents are given by month, while demand is categorized according to a demand pattern definition that is not necessarily monthly. Thus, the probabilities of weather and incidents must be aggregated across the demand patterns. The demand pattern–dependent probabilities of weather and incidents are computed on the basis of Equations 5.5 and 5.6. The equations are illustrated with numerical calculations for incorporating the effects of medium rain (weather event 1) and one-lane closure (incident event 3) probabilities into the demand pattern prevalent on Thursdays in the spring season (DP = 5) for the I-40 EB case study facility. In the equations below, j refers to a month, u to a demand pattern, and i to a weather or incident type.

$$p_w^{DP}(u, i) = \frac{\sum_{j\epsilon DP} p_w(i, j) \times N_{DP}(u, j)}{\sum_{j\epsilon DP} N_{DP}(u, j)} \tag{5.5}$$

$$\begin{aligned}p_w^{DP}(5, 1) &= \frac{\sum_{j=3}^{5} p_w(1, j) \times N_{DP}(5, j)}{\sum_{j=3}^{5} N_{DP}(5, j)}\\&= \frac{0.00505 \times 4 + 0 \times 5 + 0.01951 \times 4}{13}\\&= 0.756\%\end{aligned} \tag{5.6}$$

The probability of a base scenario is the product of the aggregated probabilities of each contributing factor. Equation 5.1 can be rewritten in the form of Equation 5.7:

$$\begin{aligned}&p_{Base}(DP = z, W = x, \text{Inc} = u)\\&\quad = p_{DP}(z) \times p_w^{DP}(x, i) \times p_{Inc}^{DP}(u, i)\end{aligned} \tag{5.7}$$

As an example, the probability of observing demand pattern 5 along with medium rain and one-lane closure conditions can be computed as shown below:

$$\begin{aligned}p_{Base}(DP = 5, W = 1, \text{Inc} = 3) &= p_{DP}(5) \times p_w^{DP}(5, 1) \times p_{Inc}^{DP}(5, 3)\\&= 0.0498 \times 0.00756 \times 0.07561\\&= 4.561 \times 10^{-5}\end{aligned}$$

Step 9: Base AND Study Period Scenarios and Probabilities for I-40 EB in Raleigh, NC

Summary of the Study Period Scenarios			
Scenario Type	Number	Percent	Probability
Demand Only	12	5.3%	2.2%
Incident Only	48	21.3%	81.1%
Weather Only	33	14.7%	7.6%
Incident and Weather	132	58.7%	9.2%
Other	0	0.0%	0.0%

Number of Base Scenarios=	225
Number of Demand Patterns=	12
Number of Days in Analysis=	261
Year=	2010

Base Scenario is Highlighted!

Base Scenario No.	Demand Pattern #	WZ and SE	Weather Label	Incident Label	Demand Multiplier	Base Scenario Probability	Study Period Scenario Probability	Number of Incidents	Number of Weather Events	Duration of the Incident (min)	Duration of the Weather Event (min)
1	7		Normal Weather	No Incident	1.07	9.84810%	0.58123%	0	0	0	0
2	10		Normal Weather	No Incident	1.02	9.70208%	0.16711%	0	0	0	0
3	4		Normal Weather	No Incident	1.08	9.65979%	0.43068%	0	0	0	0
4	1		Normal Weather	No Incident	1.00	8.84736%	0.00843%	0	0	0	0
5	7		Normal Weather	Shoulder Closure	1.07	3.72384%	6.38373%	7	0	210	0
6	4		Normal Weather	Shoulder Closure	1.08	3.70996%	4.94661%	9	0	270	0
7	10		Normal Weather	Shoulder Closure	1.02	3.46837%	5.94577%	7	0	210	0
8	3		Normal Weather	No Incident	1.07	3.34418%	0.01841%	0	0	0	0
9	12		Normal Weather	No Incident	1.13	3.23240%	0.08657%	0	0	0	0
10	11		Normal Weather	No Incident	1.09	3.22386%	0.20619%	0	0	0	0
11	8		Normal Weather	No Incident	1.13	3.19585%	0.16124%	0	0	0	0
12	9		Normal Weather	No Incident	1.18	3.19585%	0.16124%	0	0	0	0
13	5		Normal Weather	No Incident	1.14	3.13899%	0.15305%	0	0	0	0
14	6		Normal Weather	No Incident	1.20	3.13899%	0.15305%	0	0	0	0
15	2		Normal Weather	No Incident	1.01	3.10974%	0.02691%	0	0	0	0
16	1		Normal Weather	Shoulder Closure	1.00	3.00484%	4.00645%	9	0	270	0
17	8		Normal Weather	Shoulder Closure	1.13	1.21203%	2.07776%	7	0	210	0
18	9		Normal Weather	Shoulder Closure	1.18	1.21203%	2.07776%	7	0	210	0
19	5		Normal Weather	Shoulder Closure	1.14	1.21019%	1.61359%	9	0	270	0

Figure 5.2. FSG schematic showing base scenarios for I-40 case study.

In fact, the base scenarios describe the operational condition of the freeway facility and the probability associated with it. The probability of a base scenario specifies the expected portion of time that the freeway facility is subject to operating at the scenario-specified conditions. Thus, each base scenario presents an expected travel time and its associated probability. By modeling these scenarios and measuring their travel times, a discrete distribution of expected travel times is generated. This expected discrete travel time distribution is used to assess the reliability of the freeway facility. The example for the I-40 EB case study generated 225 distinct base scenarios describing the facility's operational condition. Figure 5.2 presents a schematic of the FSG output, which shows each base scenario and its probability (column with an arrow).

Study Period for Freeway Scenario Generation

While the base scenarios describe the general conditions under which the facility will operate during a study period (e.g., a weather event will occur sometime during the study period and an incident will take place sometime and somewhere on the facility), they lack the specificity that enables an analyst to model the events' effect in the FREEVAL-RL computational engine. This gives rise to the term *study period (SP) scenarios,* in which event durations are specified and adjustments to the base scenario probabilities take place. To summarize, each base scenario is associated with a unique SP scenario. The only difference in the two is the probability associated with each type (see the column to the right of the one with the arrow in Figure 5.2). This section describes the computations required to achieve the transition from a base scenario to an SP scenario, beginning with a simple example that motivates the need to develop SP scenarios.

Motivation Using a Simple Example

Facility Description

Consider a freeway facility consisting of 10 HCM segments. The reliability reporting period contains 50 workday Fridays, each of which has the same demand pattern. The study period is 3:00 to 7:00 p.m., resulting in 16 15-min analysis periods.

For simplicity, one severe weather condition and one incident are considered in the reliability reporting period: medium rain with a total duration of 600 min, and one-lane closure with a total duration of 900 min. Table 5.6 summarizes these conditions with respect to their time-wise probabilities.

The time-wise probability expresses the likelihood an event will occur in any time instance during the reliability reporting period. This probability translates into any time period that can be reported. For example, if the duration of the study period is 4 hours, then the expectation is that the event will be present for

Table 5.6. Example Time-Wise Probabilities of Event Occurrences

Event	Time-Wise Probability of Occurrence
Weather Event	
Medium rain	$\frac{600 \text{ min duration}}{50 \text{ study periods} \times 4 \text{ h/study period} \times 60 \text{ min/h}} = 0.05$
Nonsevere weather	$1 - 0.05 = 0.95$
Incident Event	
One-lane closure	$\frac{900 \text{ min duration}}{50 \text{ study periods} \times 4 \text{ h/study period} \times 60 \text{ min/h}} = 0.075$
No incident	$1 - 0.075 = 0.925$

a period of time equal to its probability times the study period duration. The term *time-wise* distinguishes this probability from other types of probabilities, such as VMT-wise, count-wise, or length-wise probabilities.

Base Scenario Development

The base scenario generation procedure is employed to generate different operational conditions on the freeway facility. These conditions are assumed to be independent. Table 5.7 summarizes the operational conditions associated with the base scenarios in this example. The base scenarios in this form are not ready to be provided to the HCM freeway facility methodology because they do not contain any of the critical event attributes that affect travel time (e.g., location, duration, start time).

The joint probabilities of these operational conditions are also time-wise. If any time instance across all study periods in the reliability reporting period is chosen, it will yield a no-severe-weather and no-incident condition (demand-only scenario) with a probability of almost 88%. Figure 5.3 depicts the probabilities associated with each base scenario.

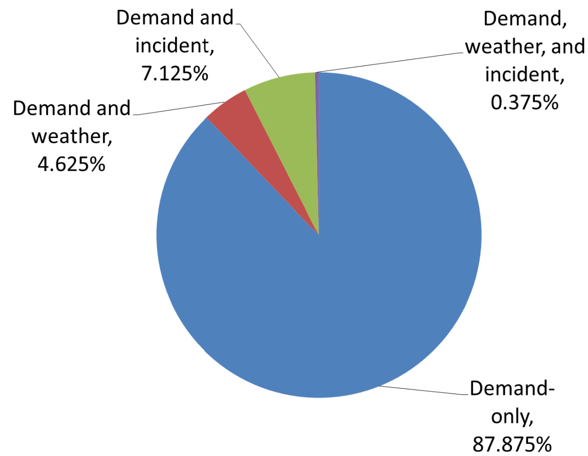


Figure 5.3. Distribution of initial scenario categories.

Study Period Scenario Development

Next, the event durations are introduced. According to historical data, the average durations are 49 min and 32 min for the one-lane closure incident and the medium rain weather event, respectively. Because the HCM freeway facilities method uses 15-min analysis periods, these average durations are rounded to 45 min and 30 min, respectively.

To accommodate the four combinations of weather and incident events being modeled, four SP scenarios are defined. Modeling these four study periods guarantees that all the operational condition characteristics are accounted for at the correct time-wise probabilities. A weight (or probability)—the SP scenario probability—is assigned to the study periods to be fully consistent with the specified likelihood of the operational conditions (base scenarios).

The objective is to determine what weight to give to each of the four SP scenarios so that the resulting travel time distribution represents the facility's prespecified operational conditions. In other words, considering the base scenario probability values p_1, p_2, p_3, p_4, and the respective durations of

Table 5.7. Example Base Scenarios

Base Scenario Number	Weather Condition	Incident Condition	Base Scenario Description	Probability
1	Nonsevere	No incident	Demand-only	$p_1 = 0.95 \times 0.925 = 0.87875$
2	Medium rain	No incident	Demand and weather	$p_2 = 0.05 \times 0.925 = 0.04625$
3	Nonsevere	One lane closed	Demand and incident	$p_3 = 0.95 \times 0.075 = 0.07125$
4	Medium rain	One lane closed	Demand, weather, and incident	$p_4 = 0.05 \times 0.075 = 0.00375$
				Sum = 1

the events and the study period, what should the study period scenario probability values π_1, π_2, π_3, and π_4 be to provide consistent time-based probabilities throughout? The study period scenario probabilities should be selected in such a way that the likelihood of the conditions modeled is identical to the base scenario probabilities.

To achieve this result, Equations 5.8 through 5.11 must be satisfied for each of the base scenarios. The logic behind each equation is to equalize the proportion of time each study period scenario should be represented, according to the base scenario probabilities, recognizing that periods of no-incident or no-severe-weather conditions exist in all four study periods.

For example, in SP scenario 2, severe weather occurs in two of the 16 analysis periods, meaning that no-incident and no-severe-weather conditions are present in the remaining 14 analysis periods. Similarly, in SP scenario 3, an incident is present in three of the 16 analysis periods and no-incident conditions are present in the remaining 13 analysis periods. Finally, in SP scenario 4, representing combined weather and incident events, the longer of the two durations (in this case, three analysis periods) determines when an event is present, while the shorter of the two durations (in this case, two analysis periods) determines how long the combined weather and incident condition occurs.

Equation 5.8 provides the equality relationship for base scenario 1, representing a demand-only condition. The probability of this scenario must equal the combined probabilities of the demand-only portions of the four study period scenarios.

$$p_1 = \left(\frac{16-0}{16}\right)\pi_1 + \left(\frac{16-2}{16}\right)\pi_2 + \left(\frac{16-3}{16}\right)\pi_3 + \left(\frac{16-3}{16}\right)\pi_4 \quad (5.8)$$

SP scenario 1 has 16 demand-only analysis periods out of 16 total analysis periods. SP scenario 2 has 14 such analysis periods out of 16, and so on. The proportion of demand-only analysis periods in each SP scenario is multiplied by that scenario's probability π_i.

Equation 5.9 provides the equality relationship for base scenario 2, representing a combined demand and severe-weather-event condition. This condition does not occur at all in SP scenarios 1, 3, or 4, and occurs during only two of the 16 analysis periods in SP scenario 2. Therefore,

$$p_2 = \left(\frac{2}{16}\right)\pi_2 \quad (5.9)$$

Similarly, a combined demand and incident condition occurs during three of the 16 analysis periods in SP scenario 3 and in one of the 16 analysis periods in SP scenario 4. A combined demand, weather, and incident condition occurs during two of the 16 analysis periods in SP scenario 4. Equations 5.10 and 5.11 give the respective equality relationships for base scenarios 3 and 4.

$$p_3 = \left(\frac{3}{16}\right)\pi_3 + \left(\frac{1}{16}\right)\pi_4 \quad (5.10)$$

$$p_4 = \left(\frac{2}{16}\right)\pi_4 \quad (5.11)$$

With four equations and four unknowns, which are π_1, π_2, π_3, and π_4, Equation 5.8 can be solved for the various π_i values, yielding the following results:

$\pi_1 = 0.23$; $\pi_2 = 0.37$; $\pi_3 = 0.37$; and $\pi_4 = 0.03$.

When those π_i values are assigned to the four specified SP scenarios, the resulting travel time distribution yields facility travel times consistent with the intended distribution of operational conditions.

Note the large difference between p_1 (88%) and π_1 (23%). This result does not mean that normal conditions have been reduced by that amount in the SP scenarios. It simply reflects that "pieces" of p_1 exist in all four SP scenarios, as indicated in the first of the four equilibrium equations (Equation 5.8). The large differences between p_2 and π_2 and between p_3 and π_3 are similarly explained: those two study period scenarios also contain many no-incident, no-severe-weather analysis periods.

The set of equilibrium equations could potentially yield infeasible results (meaning one of the resulting π_i values is negative). That could occur if the likelihood of the weather or incident event is high and the expected event duration is short. In those cases, the duration of the event should be increased, or more than one event per study period should be modeled.

Detailed Scenario Development

The final step in the scenario generation process is to develop the detailed scenarios. Weather events have two possible start times; incidents have three possible start times, three possible durations, and two possible locations. Each possible combination is assumed to occur with equal probability.

Figure 5.4 depicts one detailed scenario from each of the four study periods associated with a study period scenario. Each study period is 4 hours (or 16 analysis periods) long, consistent with the specified duration. The figure shows the expected duration and location of the weather and incident events associated with the detailed scenarios.

At this point, sufficient information is available to model the facility by using the HCM freeway facilities method, as the weather and incident events have been fully specified according to start time, duration, and affected segments. In addition, the probabilities of each detailed scenario have been determined,

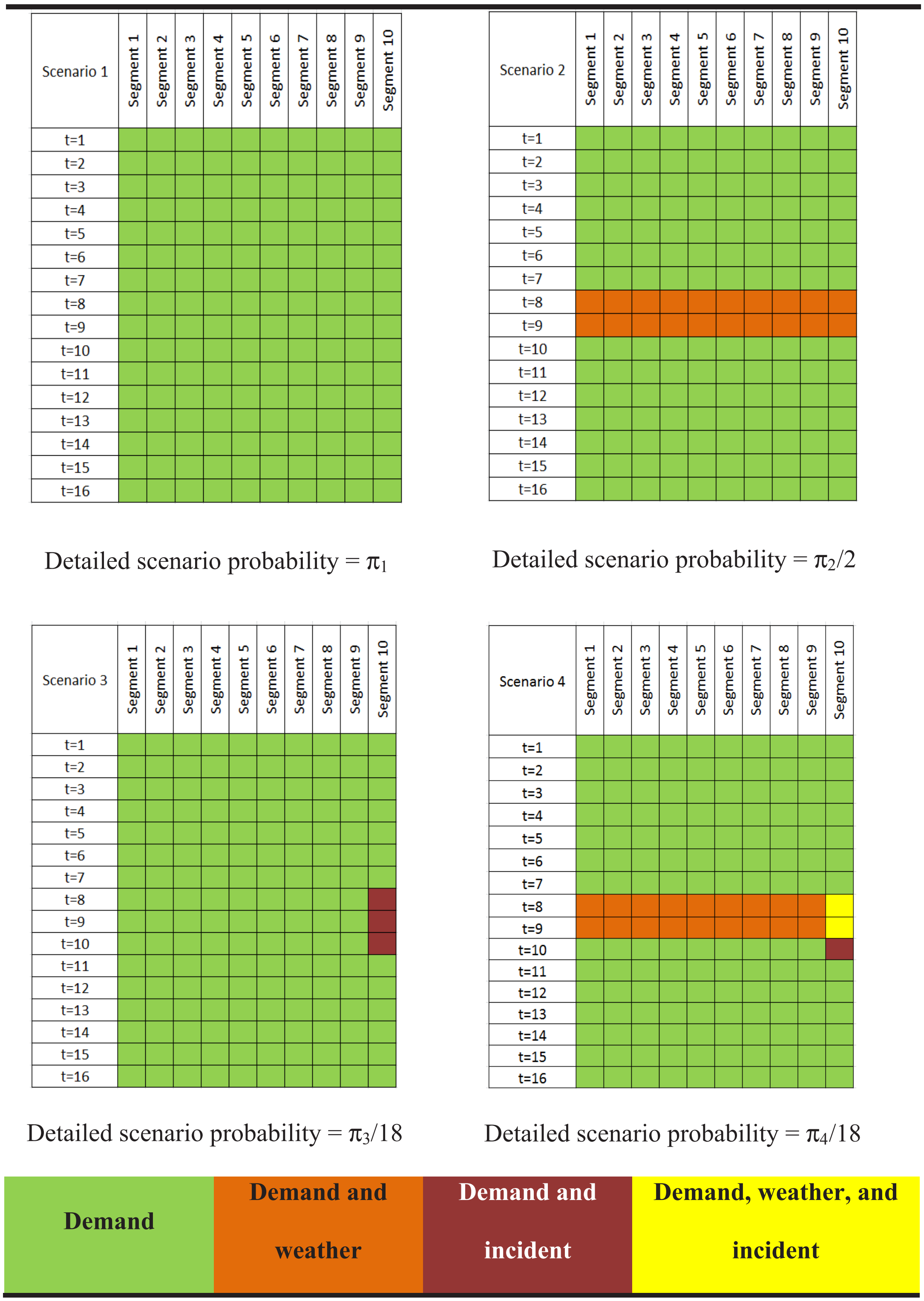


Figure 5.4. Event occurring during each analysis period of selected detailed scenarios.

Table 5.8. I-40 EB Base Scenarios and Probabilities for Demand Pattern 1

Incident Category	Weather Category: Nonsevere Weather (0) (%)	Medium Rain (1) (%)	Low Visibility (2) (%)	Light to Medium Snow (3) (%)	Light Snow (4) (%)	Sum of Probabilities (%)
No incident (0)	**8.8473650**	0.14309	0.06633	0.01666	0.44710	**9.52054**
Shoulder closure (1)	3.00484	0.05025	0.02332	0.00531	0.14825	**3.23197**
One-lane closure (2)	0.90935	0.01524	0.00707	0.00160	0.04479	**0.97805**
Two-lane closure (3)	0.23029	0.00386	0.00179	0.00040	0.01134	**0.24769**
Three-lane closure (4)	0.18409	0.00309	0.00143	0.00032	0.00906	**0.19799**
Sum of probabilities	**13.17593**	**0.21553**	**0.09995**	**0.02430**	**0.66053**	**14.17625**

allowing the resulting travel time distribution to be properly aggregated.

The final results of applying the adjusted probabilities for the I-40 EB case study are shown in Table 5.8. Note that the sum of the probabilities for the demand pattern is preserved in both cases, with only the allocation of probabilities across the 20 scenarios varying. Also, as noted earlier, the weights for the no-severe-weather, no-incident scenario decreased by a factor of 1,000 from 8.8% to 0.0084%, as shown in Table 5.9.

To model weather and incident events, the start time, duration, and location of the events on the facility should be estimated. Although the start time and precise location of an event can be determined in the latter steps of the analysis, the event duration is critical in adjusting the base scenario probability to avoid biasing the resulting distribution, as shown in the simple example above.

A special algorithm is applied to model incident and weather events inside each study period. Event duration is usually low compared with the duration of the study period. The algorithm adjusts the probabilities of base scenarios that have events shorter than their study period. All weather and incident events are modeled assuming their mean duration only. If a single mean event duration is inadequate to honor the initial event probability Π, with adjusted $\Pi > 1.0$, another event of the same duration is appended in the study period. Thus, the algorithm determines the number of events and the durations that are required to match the stated probabilities.

Algorithm Assumptions

The following assumptions are built into the event modeling algorithm:

- Incident event durations may be altered during detailed scenario generation, without altering the study period probabilities. This assumption is not overly severe, since

Table 5.9. I-40 EB Adjusted Scenario Probabilities for Demand Pattern 1

Incident Category	Weather Category: Nonsevere Weather (0) (%)	Medium Rain (1) (%)	Low Visibility (2) (%)	Light to Medium Snow (3) (%)	Light Snow (4) (%)	Sum of Probabilities (%)
No incident (0)	**0.00843**	0.88275	0.21562	0.10565	0.22294	**1.43539**
Shoulder closure (1)	4.00645	0.60302	0.27983	0.06371	0.88950	**5.84251**
One-lane closure (2)	3.63738	0.18290	0.08489	0.01919	0.53746	**4.46183**
Two-lane closure (3)	1.37323	0.03090	0.01076	0.00324	0.06802	**1.48615**
Three-lane closure (4)	0.87098	0.02470	0.00860	0.00259	0.04350	**0.95037**
Sum of probabilities	**9.89649**	**1.72426**	**0.59970**	**0.19437**	**1.76142**	**14.17625**

the three possible incident durations are selected to be at, below, and above the originally assumed mean duration.

- Modeling in FREEVAL requires all events to be rounded to the nearest 15-min increment, to be consistent with HCM analysis period durations. This process introduces some errors and bias to the reliability calculations; however, the algorithm accounts for this bias and eliminates its effects.

Scenario Categories

In general, scenarios are divided into four categories:

- Demand-only (normal condition) scenarios (Category 1 scenarios);
- Weather-only scenarios (Category 2 scenarios);
- Incident-only scenarios (Category 3 scenarios); and
- Combined incident and weather scenarios (Category 4 scenarios).

This categorization is needed to execute the probability adjustment procedure in the generation of SP scenarios. In general, the first category usually has a high probability of occurrence. As an example, Category 1 scenarios have a probability of about 64% in the I-40 EB case study. Demand patterns are modeled using the demand adjustment factors (DAFs). Each scenario (basic, study period, and detailed) has an associated demand multiplier (DM) that applies to all segments and time periods. To model the effects of weather and incident events, appropriate capacity adjustment factors (CAFs) and free-flow speed adjustment factors (SAFs) are applied to the affected segments and time periods. For incidents, the number of open lanes should also be adjusted according to the type of incident. The remaining sections focus on (1) the generation of base scenarios in the FSG; (2) the challenge of modeling events in the study periods, by mapping and changing the probability vector for the base scenarios; and (c) detailed scenarios that are entered into the computational engine FREEVAL-RL.

Subsets of Base Scenarios

In a facility with *N* demand patterns, all base scenarios can be divided into *N* subsets. The subsets are mutually exclusive, and their union covers all base scenarios. The methodology proposed for adjusting SP scenario probabilities applies to each subset separately. Table 5.10 presents one such subset associated with demand pattern 1 for the I-40 EB case study (the sum of probabilities is 14.18% as per Table 5.8 and Table 5.9).

Conceptual Approach

The methodology for the SP scenario probability adjustment creates weather or incident events in the study period with a predetermined duration. The remaining time periods in that study period actually describe another scenario from Table 5.10 (usually the parent scenario, base scenario 4). Therefore, each SP scenario is associated with more than one base scenario.

Figure 5.5 depicts an example in which an SP scenario represents three base scenario categories, demand-only (during t_1 and t_4), demand and weather (during t_3), and demand, weather, and incident (during t_2). If the probability of the occurrence of this SP scenario is given as Π, then Equations 5.12 through 5.15 give the relationships between the probabilities of base and SP scenarios.

Category 1 (Demand Only)

$$\text{Base Scenario's Probability} = \prod \times \left(\frac{t_1 + t_4}{\text{SP}}\right) \tag{5.12}$$

Category 2 (Weather Only)

$$\text{Base Scenario's Probability} = \prod \times \left(\frac{t_3}{\text{SP}}\right) \tag{5.13}$$

Category 3 (Incident Only)

$$\text{Base Scenario's Probability} = 0 \tag{5.14}$$

Category 4 (Weather and Incident)

$$\text{Base Scenario's Probability} = \prod \times \left(\frac{t_2}{\text{SP}}\right) \tag{5.15}$$

As shown in the Equations 5.12 through 5.15, the relationship between the base and SP scenario probabilities is one-to-one. In this method, the base scenarios' probabilities are known and the SP scenario probabilities (Π) are calculated.

Core SP Scenario Generation: Probability Adjustments

The core of the methodology relies on adjusting event durations. SP scenarios, with their adjusted probabilities, provide a freeway system operation similar to the base scenarios from a travel time perspective. The methodology consists of 10 steps. The data presented in Table 5.10 are used throughout this section as an example for following the steps in the methodology. Figure 5.6 shows the methodology's process flow.

Step 1: Select the Desired Subset of Base Scenarios Associated with a Specific Demand Pattern

All base scenarios associated with demand pattern 1 are grouped into one subset. The data in Table 5.11 shows five

Table 5.10. Subset of Base Scenarios Associated with Demand Pattern 1

Base Scenario No.	Demand Pattern No.	Weather Label	Incident Label	Probability of Base Scenario (%)	Scenario Category No.
4	1	Nonsevere weather	No incident	8.84736	1
16	1	Nonsevere weather	Shoulder closure	3.00484	3
28	1	Nonsevere weather	One-lane closure	0.90935	3
29	1	Light snow	No incident	0.44710	2
42	1	Nonsevere weather	Two-lane closure	0.23029	3
45	1	Nonsevere weather	Three-lane closure	0.18409	3
48	1	Light snow	Shoulder closure	0.14825	4
49	1	Medium rain	No incident	0.14309	2
68	1	Low visibility	No incident	0.06633	2
74	1	Medium rain	Shoulder closure	0.05025	4
77	1	Light snow	One-lane closure	0.04479	4
88	1	Low visibility	Shoulder closure	0.02332	4
96	1	Light to medium snow	No incident	0.01666	2
99	1	Medium rain	One-lane closure	0.01524	4
104	1	Light snow	Two-lane closure	0.01134	4
117	1	Light snow	Three-lane closure	0.00906	4
120	1	Low visibility	One-lane closure	0.00707	4
128	1	Light to medium snow	Shoulder closure	0.00531	4
138	1	Medium rain	Two-lane closure	0.00386	4
146	1	Medium rain	Three-lane closure	0.00309	4
163	1	Low visibility	Two-lane closure	0.00179	4
164	1	Light to medium snow	One-lane closure	0.00160	4
166	1	Low visibility	Three-lane closure	0.00143	4
203	1	Light to medium snow	Two-lane closure	0.00040	4
209	1	Light to medium snow	Three-lane closure	0.00032	4

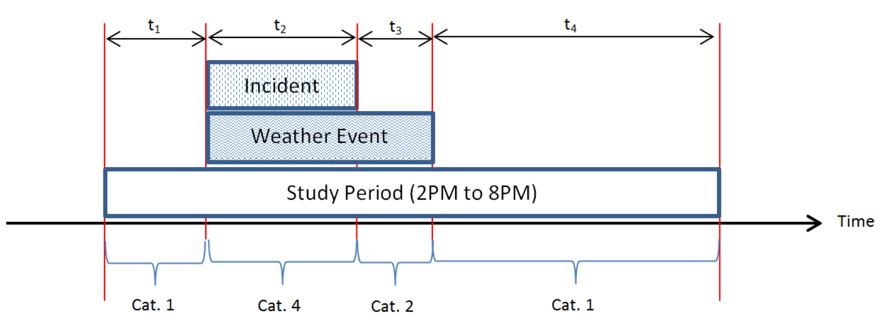


Figure 5.5. Typical study period with incident and weather event (Category 4 scenario).

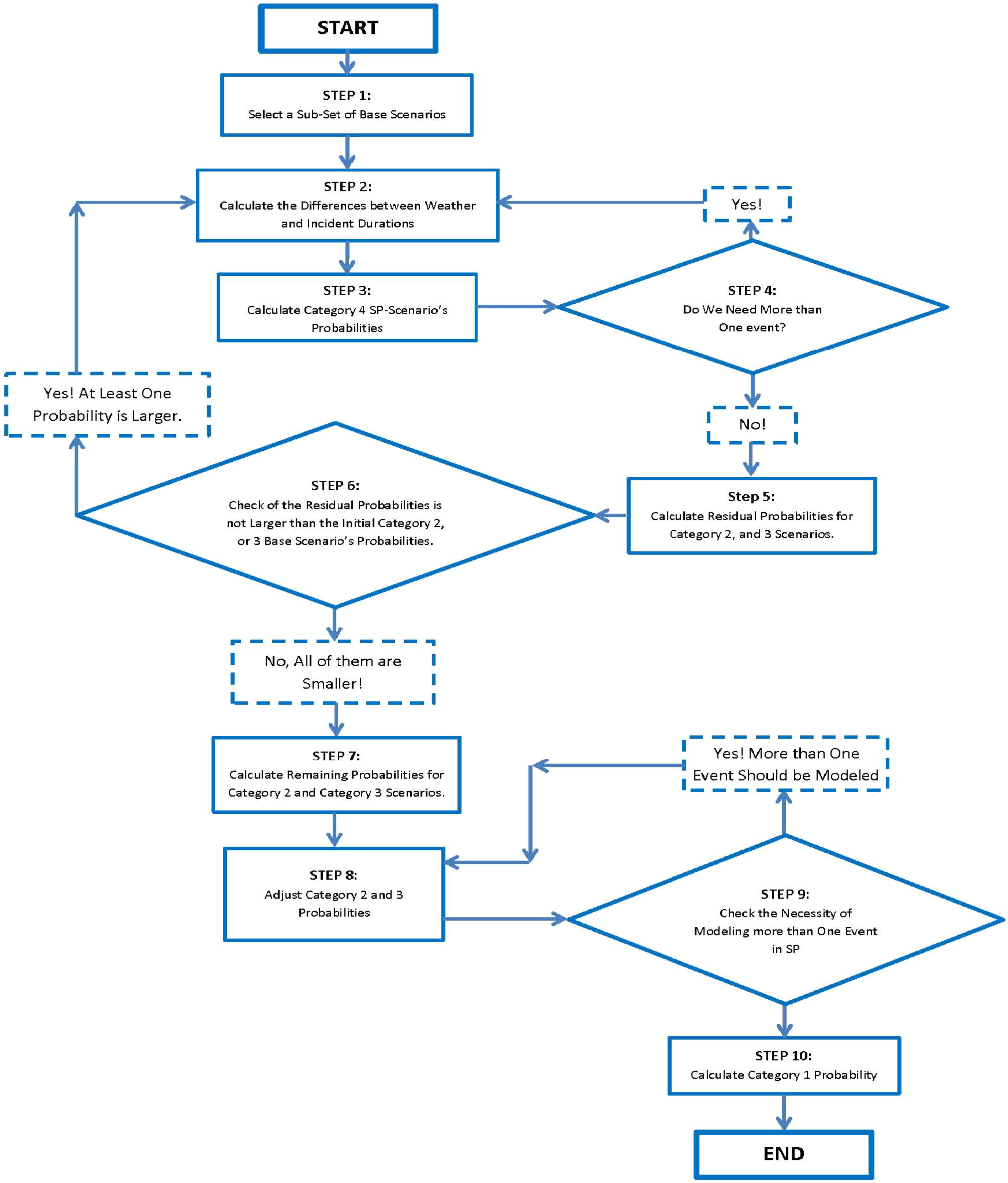


Figure 5.6. Probability adjustment methodology for SP scenarios.

Table 5.11. Combinations of Weather and Incidents Associated with Demand Pattern 1 and Their Probabilities

Incident Category (j)	Weather Category (i)					Sum of Probabilities (%)
	Nonsevere Weather (0) (%)	Medium Rain (1) (%)	Low Visibility (2) (%)	Light to Medium Snow (3) (%)	Light Snow (4) (%)	
No incident (0)	8.847365	0.14309	0.06633	0.01666	0.44710	9.52054
Shoulder closure (1)	3.00484	0.05025	0.02332	0.00531	0.14825	3.23197
One-lane closure (2)	0.90935	0.01524	0.00707	0.00160	0.04479	0.97805
Two-lane closure (3)	0.23029	0.00386	0.00179	0.00040	0.01134	0.24769
Three-lane closure (4)	0.18409	0.00309	0.00143	0.00032	0.00906	0.19799
Sum of probabilities	13.17593	0.21553	0.09995	0.02430	0.66053	14.17625

weather and five incident categories. The incident categories are no incident, shoulder closure, one-lane closure, two-lane closure, and three-lane closure. Weather events are nonsevere weather, medium rain, low visibility, light to medium snow, and light snow. The parent scenario of this subset is the first base scenario in Table 5.10 (base scenario 4). The parent scenario has a relatively large probability of occurrence compared with other scenarios. Table 5.11 presents combinations of weather and incident events for the specified subset along with their probabilities.

As shown in Table 5.11, the sum of probabilities for all scenarios is 14.176%. Therefore, the sum of the adjusted probabilities for the SP scenarios must also be 14.176%. Different categories of base scenarios are shown with different background colors. Green represents Category 1, blue represents Categories 2 and 3, and pink represents Category 4.

Step 2: Calculate the Time Differences Between Weather and Incident Event Durations

Denote τ_i^w as the duration of weather event i, and τ_j^{inc} as the duration of incident type j. The indices for each weather and incident categories are shown in parentheses in Table 5.11. Modeling any weather or incident event requires its duration to be rounded to the nearest 15-min increment. In this section, "Round (τ)" symbolizes the rounded value of τ to its nearest 15-min value.

According to the definition of Category 4 base scenarios, the effects of weather and incidents apply to the freeway facility with the same duration. In reality, they might have different durations. Therefore, the durations of weather and incident events are compared in this step and the differences are calculated. For each Category 4 scenario, ω_{ij} and Δ_{ij} are defined on the basis of Equations 5.16 and 5.17.

$$\omega_{ij} = \text{Min}\left(\text{Round}\left(\tau_i^w\right), \text{Round}\left(\tau_j^{\text{inc}}\right)\right) \tag{5.16}$$

$$\Delta_{ij} = \left|\text{Round}\left(\tau_i^w\right) - \text{Round}\left(\tau_j^{\text{inc}}\right)\right| \tag{5.17}$$

Thus, ω_{ij} represents the time that both weather and incident events occur in Category 4 base scenarios. Table 5.12 and Table 5.13 present the durations of weather and incident events for the I-40 EB case study.

Table 5.14 and Table 5.15 show the values of ω_{ij} and Δ_{ij} for the I-40 EB case study, based on Equations 5.16 and 5.17.

Table 5.12. I-40 EB Duration of Different Weather Categories

Weather Category	Expected Duration (min)	Rounded Value to Nearest 15-min Increment
Medium rain	42.9	45
Low visibility	57.2	60
Light to medium snow	46.6	45
Light snow	134.3	135

Table 5.13. I-40 EB Duration of Different Incident Categories

Incident Category	Expected Duration (min)	Rounded Value to Nearest 15-min Increment
Shoulder closure	32	30
One-lane closure	34	30
Two-lane closure	53	60
Three-lane closure	69	75

Table 5.14. I-40 EB Calculated ω_{ij} Values

Incident Category	Medium Rain (1)	Low Visibility (2)	Light to Medium Snow (3)	Light Snow (4)
Shoulder closure (1)	30	30	30	30
One-lane closure (2)	30	30	30	30
Two-lane closure (3)	45	60	45	60
Three-lane closure (4)	45	60	45	75

Note: Calculated values are in minutes.

Table 5.15. I-40 EB Calculated Δ_{ij} Values

Incident Category	Medium Rain (1)	Low Visibility (2)	Light to Medium Snow (3)	Light Snow (4)
Shoulder closure (1)	15	30	15	105
One-lane closure (2)	15	30	15	105
Two-lane closure (3)	15	0	15	75
Three-lane closure (4)	30	15	30	60

Note: Calculated values are in minutes.

Step 3: Calculate Category 4 SP Scenario Probability

Denote p_{ij} and π_{ij} as the probabilities of base scenarios and SP scenarios, respectively. The duration of the study period is symbolized by SP. If there is only a single weather event coinciding with a single incident event in the SP scenario then the relationship between the SP scenario's probability and the base scenario's probability is in the form of Equation 5.18:

$$p_{ij} = \pi_{ij} \times \left(\frac{\omega_{ij}}{\mathrm{SP}}\right) \qquad (5.18)$$

Equation 5.18 defines a one-to-one relationship between the SP scenario and base scenario probabilities. It indicates that the probability of a base scenario is the proportion of time that has the same condition in the SP, multiplied by the probability of the SP scenario. Although the condition immediately after the event is not completely the same as that represented by the parent (nonsevere weather and no incident) scenario (e.g., the impact of wet pavement after a rain event has ended), that effect is ignored in the method. Nevertheless, the bias imposed by this assumption is considered negligible. Equation 5.19 gives the probability of the SP scenarios as a function of the probability of the base scenarios.

$$\pi_{ij} = p_{ij} \times \left(\frac{\mathrm{SP}}{\omega_{ij}}\right) \qquad (5.19)$$

Step 3 calculates π_{ij} values for all Category 4 scenarios, as illustrated in Table 5.16.

Table 5.16. Adjusted Probabilities (π_{ij}) for Category 4 Scenarios

Incident Category (j)	Weather Category (i): Normal Weather (0)	Medium Rain (1) (%)	Low Visibility (2) (%)	Light to Medium Snow (3) (%)	Light Snow (4) (%)	Sum of Probabilities (%)
No incident (0)	N/A	N/A	N/A	N/A	N/A	**N/A**
Shoulder closure (1)	N/A	0.60302	0.27983	0.06371	1.77900	**2.72556**
One-lane closure (2)	N/A	0.18290	0.08489	0.01919	0.53746	**0.82444**
Two-lane closure (3)	N/A	0.03090	0.01076	0.00324	0.06802	**0.11434**
Three-lane closure (4)	N/A	0.02470	0.00860	0.00259	0.04350	**0.08202**
Sum of probabilities	**N/A**	**0.87426**	**0.41448**	**0.09218**	**1.90197**	**3.7423**

Note: N/A = not applicable.

Step 4: Check the Necessity for Modeling More than One Event in Category 4 Scenarios

The sum of all probabilities generated in Step 3 for Category 4 scenarios should be less than the total sum of the base scenario probabilities. Otherwise, the SP scenarios must model more than one event (or overall duration) per study period as the only possible option to address this problem. Equation 5.20 should thus hold for proceeding with the methodology with no change in event durations (i.e., Step 5).

$$\sum_{\substack{i=1 \text{ to } 4 \\ j=1 \text{ to } 4}} \pi_{ij} < \sum_{\substack{i=0 \text{ to } 4 \\ j=0 \text{ to } 4}} p_{ij} \tag{5.20}$$

In Equation 5.20, i and j represent the weather and incident category indices, respectively. For the I-40 EB case study, the total sum of probabilities of the subset associated with demand pattern 1 is 14.176%, which is equal to the sum of all base scenario probabilities. The sum of probabilities generated in Step 3 is 3.74%. The condition for continuing the methodology holds on the basis of Equation 5.20:

$$3.74\% < 14.176\%$$

If the constraint in Equation 5.20 is not met, then the solution to the problem lies in modeling more than one incident and weather event simultaneously. In that case, the process of modeling more than one event should be followed (i.e., increase the values of ω_{ij}), and Steps 2 and 3 should be repeated to make sure that the sum of all probabilities is low enough to warrant proceeding with the rest of the methodology. Differences between durations of weather events and incidents should also be investigated. In some cases, the problem is solved by repeating the shortest event (which is usually the incident). This process models two incidents concurrent with one weather event. If any such changes are made, Steps 2 and 3 should be repeated.

Step 5: Calculate Residual Probabilities for Category 2 and 3 Scenarios

Residual probabilities are imposed by the differences in durations of the weather events and incidents in Category 4 scenarios. In Step 3, the study period was modeled with weather events and incidents, together with a duration of ω_{ij} and a probability π_{ij}. However, because weather events and incidents are likely to have different durations, the effect of the longer of the two events should be modeled to maintain accuracy.

Denote W_i as a Category 4 scenario when the rounded weather event (i) duration is greater than the rounded incident's duration and I_j as a Category 4 scenario when the incident (j) duration is greater than the weather event duration. Finally, a flag N is assigned whenever the rounded incident and weather durations are equal. For type N scenarios, the residual probabilities need not be computed. This step focuses only on type W and type I scenarios. Table 5.17 shows the various flags associated with the different weather event and incident combinations for the I-40 EB case study.

In this step, a portion of the probability of each weather-only (Category 2) scenario is assigned to the cell in the same column as the W-flagged scenarios, and a portion of the probability of incident-only (Category 3) scenarios is assigned to each cell in the same row as the I-flagged scenarios. The reason is that the generated SP scenarios in Step 3 not only represent Category 4 base scenarios, but some of them also represent Categories 2 and 3.

Denote α_{ij} as an indicator variable, where

$$\alpha_{ij} = \begin{cases} 1, & \text{if the flag of scenario with weather type } i \\ & \text{and incident type } j \text{ is } W; \\ 0, & \text{Otherwise.} \end{cases} \tag{5.21}$$

Table 5.17. I-40 EB Flags for Weather Events and Incident Scenarios

	Weather Category (*i*)				
Incident Category (*j*)	**Normal Weather (0)**	**Medium Rain (1)**	**Low Visibility (2)**	**Light to Medium Snow (3)**	**Light Snow (4)**
No incident (0)	N/A	N/A	N/A	N/A	N/A
Shoulder closure (1)	N/A	W_1	W_2	W_3	W_4
One-lane closure (2)	N/A	W_1	W_2	W_3	W_4
Two-lane closure (3)	N/A	I_3	N	I_3	W_4
Three-lane closure (4)	N/A	I_4	I_4	I_4	W_4

Note: N/A = not applicable.

Denote β_{ij} as an indicator variable, where

$$\beta_{ij} = \begin{cases} 1, & \text{if the flag of scenario with weather type } i \\ & \text{and incident type } j \text{ is } I; \\ 0, & \text{Otherwise.} \end{cases} \tag{5.22}$$

In each column in Table 5.18, the probability residual in Category 4 scenarios assigned to Category 2 scenarios is calculated on the basis of Equation 5.23. Denote π'_i as this residual probability:

$$\pi'_i = \sum_{j=1}^{4} \pi_{ij} \times \alpha_{ij} \times \left(\frac{\Delta_{ij}}{SP}\right) \tag{5.23}$$

In each column in Table 5.18, the probability residual in the Category 4 scenarios assigned to Category 3 scenarios is calculated on the basis of Equation 5.24. Denote π''_j as this residual probability.

$$\pi''_j = \sum_{i=1}^{4} \pi_{ij} \times \beta_{ij} \times \left(\frac{\Delta_{ij}}{SP}\right) \tag{5.24}$$

The purpose of using α_{ij} and β_{ij} is to filter the scenarios that have *W* or *I* flags. Table 5.18 presents the calculated values for residual probabilities in Step 5 for the I-40 EB case study. They indicate that Category 4 scenarios already account for a portion of Category 2 or 3 scenarios. These residual probabilities should therefore be subtracted from the initial base scenario probabilities.

Step 6: Check that the Residual Probabilities Are Lower than Category 2 and 3 Initial Base Scenario Probabilities

If π'_i and π''_j are greater than the probability of Category 2 and 3 scenarios, that means the impact of time difference between the duration of the weather event and the duration of the incident (Δ_{ij}) is larger than the impact of the expected weather-only or incident-only base scenario. That means the shorter event must be modeled with a longer duration in Step 3, and the procedure needs to be restarted again from Step 3. To proceed to the next step, Equation 5.25 and Equation 5.26 must hold.

$$\pi''_j < p_{ij}, i = 0 \tag{5.25}$$

$$\pi'_i < p_{ij}, j = 0 \tag{5.26}$$

For the I-40 EB case study, substituting in Equation 5.25 for Category 2 scenarios gives

- 0.5488% < 0.4471% (for light snow);
- 0.0035% < 0.0167% (for light to medium snow);
- 0.0304% < 0.0663% (for low visibility); and
- 0.0328% < 0.1431% (for medium rain).

It is evident in the top equation above that the condition has not been satisfied. For Category 3 scenarios, substituting in Equation 5.26 gives

- 0.00103% < 0.03191% (for two-lane closure); and
- 0.00082% < 0.02548% (for three-lane closure).

Given these results, two shoulder closures must be modeled besides light snow in the Category 4 scenario associated with these two events. Table 5.19 shows the resulting new set of probabilities for SP scenarios.

Now the condition in Equation 5.26 holds, allowing the procedure to move on:

0.3635% < 0.4471% (for light snow)

Table 5.18. Residual Probabilities for Incident-Only or Weather-Only Scenarios

Incident Category (*j*)	Weather Category (*i*)					Sum of Probabilities (%)
	Normal Weather (0) (%)	Medium Rain (1) (%)	Low Visibility (2) (%)	Light to Medium Snow (3) (%)	Light Snow (4) (%)	
No incident (0)	N/A	0.03275	0.03039	0.00345	0.54880	**0.61540**
Shoulder closure (1)	N/A	0.60302	0.27983	0.06371	1.77900	**N/A**
One-lane closure (2)	N/A	0.18290	0.08489	0.01919	0.53746	**N/A**
Two-lane closure (3)	0.00142	0.03090	0.01076	0.00324	0.06802	**N/A**
Three-lane closure (4)	0.00263	0.02470	0.00860	0.00259	0.04350	**N/A**
Sum of probabilities	**0.00405**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**

Note: N/A = not applicable.

Table 5.19. Corrected Residual Probabilities for Category 2 Scenarios

Incident Category (*j*)	Weather Category (*i*)					Sum of Probabilities (%)
	Normal Weather (0) (%)	Medium Rain (1) (%)	Low Visibility (2) (%)	Light to Medium Snow (3) (%)	Light Snow (4) (%)	
No incident (0)	N/A	0.03275	0.03039	0.00345	0.36349	**0.4300**
Shoulder closure (1)	N/A	0.60302	0.27983	0.06371	0.88950	**N/A**
One-lane closure (2)	N/A	0.18290	0.08489	0.01919	0.53746	**N/A**
Two-lane closure (3)	0.00142	0.03090	0.01076	0.00324	0.06802	**N/A**
Three-lane closure (4)	0.00263	0.02470	0.00860	0.00259	0.04350	**N/A**
Sum of probabilities	**0.00405**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**

Note: N/A = not applicable.

Note that after modeling two incidents in the Category 4 scenario associated with light snow and shoulder closure, the ω_{ij} and Δ_{ij} values should be updated for that specific scenario.

Step 7: Calculate Remaining Probabilities of Category 2 and 3 Scenarios

To model events in Category 2 and 3 scenarios, their base scenario remaining probabilities (in addition to the Category 4 residuals) should be calculated. These probabilities show the portion of base scenario probabilities that is not modeled in Category 4 SP scenarios. In the next step, an adjustment of the SP Category 2 and 3 scenario probabilities will be calculated. Equations 5.27 and 5.28 give the remaining probabilities for Category 2 and 3 scenarios.

Remainder Probabilities for

$$\text{Weather Only Scenarios} = p_{ij} - \pi'_i \qquad (5.27)$$

Remainder Probabilities for

$$\text{Incident Only Scenarios} = p_{ij} - \pi''_j \qquad (5.28)$$

Checking the probabilities in Step 6 ensures that the probabilities are positive in Step 7. Table 5.20 presents the remainder probabilities for Category 2 and 3 scenarios.

Step 8: Adjust Category 2 and 3 Probabilities

In Step 7, the base scenario remainder probabilities of Category 2 or 3 scenarios were calculated. In Step 8, those probabilities are adjusted on the basis of Equation 5.29 to

Table 5.20. Remainder Probability of Incident-Only (Category 3) and Weather-Only (Category 2) Scenarios

Incident Category (*j*)	Weather Category (*i*)					Sum of Probabilities (%)
	Normal Weather (0) (%)	Medium Rain (1) (%)	Low Visibility (2) (%)	Light to Medium Snow (3) (%)	Light Snow (4) (%)	
No incident (0)	N/A	0.11034	0.03594	0.01321	0.08360	**0.24309**
Shoulder closure (1)	0.60302	N/A	N/A	N/A	N/A	N/A
One-lane closure (2)	0.18290	N/A	N/A	N/A	N/A	N/A
Two-lane closure (3)	0.03090	N/A	N/A	N/A	N/A	N/A
Three-lane closure (4)	0.02470	N/A	N/A	N/A	N/A	N/A
Sum of probabilities	**0.95186**	N/A	N/A	N/A	N/A	N/A

Note: N/A = not applicable.

Table 5.21. Adjusted Probabilities for Category 2, 3, and 4 Scenarios

Incident Category (*j*)	Weather Category (*i*)					Sum of Probabilities (%)
	Normal Weather (0) (%)	Medium Rain (1) (%)	Low Visibility (2) (%)	Light to Medium Snow (3) (%)	Light Snow (4) (%)	
No incident (0)	N/A	0.88275	0.21562	0.10565	0.22294	1.43539
Shoulder closure (1)	36.05807	0.60302	0.27983	0.06371	0.88950	37.89413
One-lane closure (2)	10.91215	0.18290	0.08489	0.01919	0.53746	11.73660
Two-lane closure (3)	1.37323	0.03090	0.01076	0.00324	0.06802	1.48615
Three-lane closure (4)	0.87098	0.02470	0.00860	0.00259	0.04350	0.95037
Sum of probabilities	49.22287	1.72426	0.59970	0.19437	1.76142	53.49420

Note: N/A = not applicable.

generate SP scenario probabilities for Categories 2 and 3. Because p_{ij} is the remaining probability in Step 7, the probability of a Category 2 scenario is computed by using Equation 5.29.

$$\pi_{i0} = p_{i0} \times \tau_i^w \left(\frac{SP}{\text{Round}\left(\tau_i^w\right)} \right) \tag{5.29}$$

The same process is used to calculate the probability of Category 3 scenarios using Equation 5.30.

$$\pi_{0j} = p_{0j} \times \left(\frac{SP}{\text{Round}\left(\tau_j^{\text{inc}}\right)} \right) \tag{5.30}$$

After applying Step 8 (and confirming in Step 9 that no further changes in the number of modeled events are needed), the remaining probabilities are assigned to the Category 1, or normal condition scenario. Table 5.21 shows the adjusted probabilities for Category 2 and 3 scenarios for the I-40 EB case study.

Step 9: Check the Necessity of Modeling More than One Event per Study Period in Category 2 and 3 Scenarios

As shown in Table 5.21, the overall sum of probabilities, excluding Category 1, is 53.49% which is greater than 14.18%, the sum of the base scenario probabilities. Thus, some Category 2 or 3 scenarios need to have more than one event occur to decrease their probabilities. Based on Equations 5.29 and 5.30, if the event duration increases, then the corresponding SP scenario probability will decrease.

A rational criterion for selecting scenarios in which to model more than one event is their current generated probabilities. In Table 5.22, some incident-only scenarios have relatively large probabilities. In the I-40 EB case study, the

Table 5.22. Adjusting Incident-Only Scenarios to Have Two Incidents (Red Cells)

Incident Category (*j*)	Weather Category (*i*)					Sum of Probabilities (%)
	Normal Weather (0) (%)	Medium Rain (1) (%)	Low Visibility (2) (%)	Light to Medium Snow (3) (%)	Light Snow (4) (%)	
No incident (0)	N/A	0.88275	0.21562	0.10565	0.22294	1.43539
Shoulder closure (1)	4.00645	0.60302	0.27983	0.06371	0.88950	5.84251
One-lane closure (2)	3.63738	0.18290	0.08489	0.01919	0.53746	4.46183
Two-lane closure (3)	1.37323	0.03090	0.01076	0.00324	0.06802	1.48615
Three-lane closure (4)	0.87098	0.02470	0.00860	0.00259	0.04350	0.95037
Sum of probabilities	9.89649	1.72426	0.59970	0.19437	1.76142	14.16781

Note: N/A = not applicable.

Table 5.23. Final Adjusted Probabilities for Demand Pattern 1, I-40 EB Case Study

Incident Category (*j*)	Weather Category (*i*): Normal Weather (0) (%)	Medium Rain (1) (%)	Low Visibility (2) (%)	Light to Medium Snow (3) (%)	Light Snow (4) (%)	Sum of Probabilities (%)
No incident (0)	0.00843	0.88275	0.21562	0.10565	0.22294	**1.43539**
Shoulder closure (1)	4.00645	0.60302	0.27983	0.06371	0.88950	**5.84251**
One-lane closure (2)	3.63738	0.18290	0.08489	0.01919	0.53746	**4.46183**
Two-lane closure (3)	1.37323	0.03090	0.01076	0.00324	0.06802	**1.48615**
Three-lane closure (4)	0.87098	0.02470	0.00860	0.00259	0.04350	**0.95037**
Sum of probabilities	**9.89649**	**1.72426**	**0.59970**	**0.19437**	**1.76142**	**14.17625**

Category 3 (incident-only) scenarios are the targets. Increasing the duration of the incident event in two scenarios that are shown in the cells with the red background in Table 5.22 brings the sum of the probabilities to less than 14.18%. The red cells show the scenarios where more than one incident is modeled consecutively, which is equivalent to longer incident duration. Nine shoulder closures and three one-lane closures are modeled in red cells.

Step 10: Calculate Category 1 Scenario Probability

The difference between the sum of probabilities of base scenarios and the current sum of probabilities should be assigned to the Category 1 (parent) scenario. Table 5.23 presents the adjusted probabilities for all SP scenarios for demand pattern 1 in the I-40 EB case study.

This set of adjusted probabilities is guaranteed to generate an unbiased travel time distribution. Some other assumptions, such as using the average duration of events, could still impose some error and bias into the analysis. These issues are listed in the future work section of this chapter. In general, the use of this methodology will result in a decrease in the probabilities of Category 1 scenarios from the base scenario values, and increase the probabilities of scenarios with any events, as evident from the summary results in Table 5.24, which summarize the combined results of applying the methodology across all 12 demand patterns.

Given the best information available to the research team, a total of 225 base scenarios were generated for the I-40 EB case study in the FSG. The 225 SP scenarios were used to generate 2,508 detailed scenarios. By grouping similar scenarios together, the total number of scenarios was reduced to 2,058 for modeling in FREEVAL-RL, as explained next.

Table 5.24. Comparison of Base and Study Period Scenario Probabilities

Statistic	Base Scenarios	SP Scenarios
Number of scenarios	225	225
Probability of Category 1 scenarios	63.64%	2.15%
Probability of Category 2 scenarios	1.86%	7.57%
Probability of Category 3 scenarios	33.56%	81.08%
Probability of Category 4 scenarios	0.94%	9.20%

Detailed Freeway Scenario Generation

As discussed in the base scenario generation section of this chapter, the travel time distribution generated by this methodology expresses the expected variation in travel time for the conditions defined by the base scenarios. Therefore, variations in event duration, start time, and location should be incorporated into the FSG methodology. Certain predefined values of these parameters are varied in the scenarios to capture their effect on the expected travel time distribution.

Specifically, incident impacts on freeway facilities are sensitive to the facility geometry (e.g., number of lanes, segment type, and segment length) as well as the prevailing demand level. Clearly, the effect of an incident on travel time can vary depending on the facility level of service, with higher impacts anticipated when the facility is operating near capacity. Thus, to capture the real effect of an incident on the freeway facility, the event's location, start time, and duration should be allowed to vary. Two possible start times are assumed for the incident, along with three possible durations and three possible locations along the facility.

Weather events, however, are assumed to affect the entire facility at once. Thus, the two principal weather parameters in developing detailed scenarios are the event start time and duration. Two possible start times are assumed, along with one possible duration.

Detailed Scenario Probabilities

In computing the detailed scenario probabilities, the system operator's point of view is taken into consideration when developing the travel time distribution. What the system operator is interested in is the aggregate performance of the facility over each 15-min analysis period during the reliability reporting period.

Referring back to the final adjusted probabilities in Table 5.23, the Category 1 probability for demand pattern 1 is about 0.0084%. Since the duration of the study period in the case study is 6 hours, or 24 analysis periods, the facility travel time in each 15 min for the Category 1 scenario is given a probability equal to 0.0084%/24 = 0.00035%.

For a Category 2 scenario—for example, a medium rain event—the probability is computed as 0.8828%/(2 × 24) = 0.0184%. The reason for dividing by 2 is that this scenario will be executed twice in FREEVAL-RL, once with the event at the start of the study period, and again with the event in the middle of the study period.

For a Category 3 scenario—say a shoulder closure incident—the probability is computed as 4.006%/(2 × 3 × 3 × 24) = 0.00927%. The reason for dividing by 18 is that the shoulder closure will be modeled 18 times in FREEVAL-RL, with three different locations, three durations, and two start times.

For a Category 4 scenario—say shoulder closure with medium rain—the probability is computed as 0.603%/(2 × 3 × 3 × 24) = 0.0014%. The reason for dividing by 18 is that the shoulder closure will be modeled 18 times in FREEVAL-RL, at three different locations, with three durations and two start times. Because the weather event is started at the same time that the incident is started, further division by 2 is not needed. Table 5.25 summarizes the variation in different modeling parameters in the detailed scenario generation.

Table 5.25. Modeling Parameters in FSG Methodology

Event	Factor Variations and Levels	Description
Weather	Start time Beginning of study period Middle of study period	
Incident	Start time Beginning of study period Middle of study period Location First basic segment Midpoint basic segment Last basic segment Duration 25th percentile incident duration 50th percentile incident duration 75th percentile incident duration	Incident duration follows a lognormal distribution.

Postprocessing Detailed Scenarios

Given the designation of incident types, some detailed scenarios are not feasible. This happens when a facility does not have the same number of cross-sectional lanes throughout. For example, by varying the location of incidents, the scenario could result in a total segment closure (e.g., by modeling a two-lane closure incident on a two-lane segment). These infeasible scenarios are purged from the final list of detailed scenarios, and their probabilities are reassigned proportionally to the remaining detailed scenarios on the basis of their probability of occurrence. In the I-40 EB case study, because the last basic segment has only two lanes, scenarios with two or more lanes closed cannot occur on that segment. In addition, when the variance and mean of incidents are small, the incident durations in different scenarios can become identical after rounding to the nearest 15 min. When this happens, the two detailed scenarios can be merged and their probabilities summed. In summary, postprocessing the detailed scenarios generally reduces the number of detailed scenarios that must be evaluated in FREEVAL-RL.

Estimating the Maximum Number of Scenarios

Equation 5.31 estimates the maximum number of detailed scenarios that could be generated. Because of the merging of some demand patterns and the application of minimum thresholds for inclusion, some weather events and incidents may have a zero probability. The total number of scenarios as a function of different impacting factors is the following:

$$\begin{aligned} N = {} & N_{\text{Demand}} + [N_{\text{Demand}} \times (N_{\text{Weather}} - 1)] \\ & \times C_{\text{Weather}} + [N_{\text{Demand}} \times (N_{\text{Incidents}} - 1)] \\ & \times C_{\text{Incidents}} + [N_{\text{Demand}} \times (N_{\text{Weather}} - 1) \times (N_{\text{Incidents}} - 1)] \\ & \times C_{\text{Incidents}} \times C_{\text{Weather}} \end{aligned} \tag{5.31}$$

N denotes the total number of scenarios, while N_{Weather} and $N_{\text{Incidents}}$ are the weather categories (11) and incident categories (6) aggregated across demand patterns, respectively. Each incident category is expressed by 18 detailed scenarios ($C_{\text{Incidents}}$), and each weather scenario is doubled (C_{Weather}). With 12 default

Table 5.26. Statistics for Detailed Scenarios Generated for I-40 EB Case Study

Scenario Type	Number	Percent
Category 1 demand-only scenarios	12	0.5%
Category 2 demand and incident scenarios	648	25.8%
Category 3 demand and weather scenarios	66	2.6%
Category 4 demand, incident, and weather scenarios	1,782	71.1%
Sum	**2,508**	

demand patterns, a maximum of 22,932 detailed scenarios can be generated.

$$N = 12 + (12 \times 10 \times 2) + (12 \times 5 \times 18) + (12 \times 10 \times 5) \times 18 \times 2$$
$$= 22{,}932$$

For the I-40 EB case study, the procedure generated 2,508 detailed scenarios. Note that some scenarios can be further merged. Table 5.26 summarizes the detailed scenario statistics.

Freeway Scenario Generation Input for FREEVAL-RL

This section discusses the parameters that are passed to FREEVAL-RL for each detailed scenario. Geometry, capacity, and demand data are three basic pieces of information that FREEVAL-RL needs to analyze a facility. In this section, the research team selected a detailed scenario to use as an example. Detailed scenario 2117 from the I-40 EB case study includes a medium rain event and a two-lane closure incident. Table 5.27 shows the specification of this detailed scenario.

Table 5.27. General Information for Detailed Scenario 2117

Category	Description
Weather type	Medium rain
Weather event start time	Middle of SP
Weather event duration (min)	45
Weather event CAF	0.928
Weather event SAF	0.930
Incident type	Two-lane closure
Incident start time	Middle of SP
Incident duration (min)	60
Incident location	Midpoint of facility
Per open lane incident CAF	0.667
Incident SAF	1.00

Two items can vary by scenario: the adjusted FFS and the operational number of lanes. Different weather and incident events can change the base FFS. Therefore, by passing a free-flow speed adjustment factor (SAF), FREEVAL-RL adjusts the FFS for certain analysis periods in the study period. Also, if a detailed scenario has a lane closure, then the number of lanes is adjusted for that specific scenario on the incident segment during the analysis periods when the incident is present. Figure 5.7 depicts the number of adjusted lanes for detailed scenario 2117 in the I-40 EB case study. Segment 23 is a four-lane basic segment at the midpoint of the facility. In analysis periods 12 through 15, highlighted in red, the number of lanes for that segment is reduced to two.

Demand Adjustments

Through the detailed seed file, the FSG has access to hourly demand values for all analysis periods in a SP. The only adjustment needed is to include the daily demand multiplier for the seed SP, which is denoted by DM_{Seed}. Then, the hourly demand on segment *i,* time period *t,* for detailed scenario *k* is computed as shown in Equation 5.32.

$$(D_i^t)_k = \left(\frac{(D_i^t)_{Seed}}{DM_{Seed}}\right)(DMDP_k) \qquad (5.32)$$

Thus, in the data-rich approach, the FSG essentially passes $\left(\frac{DMDP_k}{DM_{Seed}}\right)$ to FREEVAL-RL. Figure 5.8 shows the demand multipliers for I-40 EB case study scenario 2117.

Capacity and Speed Adjustments

Modeling an incident or weather event on a freeway facility in FREEVAL-RL is done by inserting (1) its capacity adjustment factor (CAF), (2) its speed adjustment factor (SAF), and (3) in the case of a lane closure, the number of operating lanes for the segment that has the incident or lane blockage.

From a capacity perspective, the FSG determines the capacity loss resulting from closed lanes (incidents or work zones) by specifying the number of operating lanes and the period of time the reduced number of lanes are in effect. In addition, the frictional effect on the remaining open lanes is then defined as the CAF.

In addition to adjusting capacity, the free-flow speed should be adjusted for any incident or weather event. This task is done by changing the SAF in FREEVAL-RL. The literature includes no evidence that incidents affect the prevailing free-flow speed, although severe weather conditions can have a significant impact. Therefore, a default value of 1 (i.e., no adjustment) is used as the free-flow speed adjustment factor for incidents.

The FSG enables the analyst to define local CAFs and SAFs for different incidents and weather events. In the absence of

Segment Number:	1	2	3	21	22	23	24	25	26	27	28
Segment Type:	B	OFR	OFR	B	ONR	B	OFR	B	ONR	ONR	B
t=1 (2:00 PM to 2:15 PM)	3	3	3	4	4	4	4	4	4	4	4
t=2 (2:15 PM to 2:30 PM)	3	3	3	4	4	4	4	4	4	4	4
t=3 (2:30 PM to 2:45 PM)	3	3	3	4	4	4	4	4	4	4	4
t=4 (2:45 PM to 3:00 PM)	3	3	3	4	4	4	4	4	4	4	4
t=5 (3:00 PM to 3:15 PM)	3	3	3	4	4	4	4	4	4	4	4
t=6 (3:15 PM to 3:30 PM)	3	3	3	4	4	4	4	4	4	4	4
t=7 (3:30 PM to 3:45 PM)	3	3	3	4	4	4	4	4	4	4	4
t=8 (3:45 PM to 4:00 PM)	3	3	3	4	4	4	4	4	4	4	4
t=9 (4:00 PM to 4:15 PM)	3	3	3	4	4	4	4	4	4	4	4
t=10 (4:15 PM to 4:30 PM)	3	3	3	4	4	4	4	4	4	4	4
t=11 (4:30 PM to 4:45 PM)	3	3	3	4	4	4	4	4	4	4	4
t=12 (4:45 PM to 5:00 PM)	3	3	3	4	4	2	4	4	4	4	4
t=13 (5:00 PM to 5:15 PM)	3	3	3	4	4	2	4	4	4	4	4
t=14 (5:15 PM to 5:30 PM)	3	3	3	4	4	2	4	4	4	4	4
t=15 (5:30 PM to 5:45 PM)	3	3	3	4	4	2	4	4	4	4	4
t=16 (5:45 PM to 6:00 PM)	3	3	3	4	4	4	4	4	4	4	4
t=17 (6:00 PM to 6:15 PM)	3	3	3	4	4	4	4	4	4	4	4
t=18 (6:15 PM to 6:30 PM)	3	3	3	4	4	4	4	4	4	4	4
t=19 (6:30 PM to 6:45 PM)	3	3	3	4	4	4	4	4	4	4	4
t=20 (6:45 PM to 7:00 PM)	3	3	3	4	4	4	4	4	4	4	4
t=21 (7:00 PM to 7:15 PM)	3	3	3	4	4	4	4	4	4	4	4
t=22 (7:15 PM to 7:30 PM)	3	3	3	4	4	4	4	4	4	4	4
t=23 (7:30 PM to 7:45 PM)	3	3	3	4	4	4	4	4	4	4	4
t=24 (7:45 PM to 8:00 PM)	3	3	3	4	4	4	4	4	4	4	4

Figure 5.7. Operational number of lanes under detailed scenario 2117.

Time Period	DM
t=1 (2:00 PM to 2:15 PM)	1.07
t=2 (2:15 PM to 2:30 PM)	1.07
t=3 (2:30 PM to 2:45 PM)	1.07
t=4 (2:45 PM to 3:00 PM)	1.07
t=5 (3:00 PM to 3:15 PM)	1.07
t=6 (3:15 PM to 3:30 PM)	1.07
t=7 (3:30 PM to 3:45 PM)	1.07
t=8 (3:45 PM to 4:00 PM)	1.07
t=9 (4:00 PM to 4:15 PM)	1.07
t=10 (4:15 PM to 4:30 PM)	1.07
t=11 (4:30 PM to 4:45 PM)	1.07
t=12 (4:45 PM to 5:00 PM)	1.07
t=13 (5:00 PM to 5:15 PM)	1.07
t=14 (5:15 PM to 5:30 PM)	1.07
t=15 (5:30 PM to 5:45 PM)	1.07
t=16 (5:45 PM to 6:00 PM)	1.07
t=17 (6:00 PM to 6:15 PM)	1.07
t=18 (6:15 PM to 6:30 PM)	1.07
t=19 (6:30 PM to 6:45 PM)	1.07
t=20 (6:45 PM to 7:00 PM)	1.07
t=21 (7:00 PM to 7:15 PM)	1.07
t=22 (7:15 PM to 7:30 PM)	1.07
t=23 (7:30 PM to 7:45 PM)	1.07
t=24 (7:45 PM to 8:00 PM)	1.07

Figure 5.8. Demand multipliers (DMs) for I-40 EB detailed scenario 2117.

local data, HCM2010 default CAFs for different types of weather and incidents can be substituted. When generating the combined capacity drop for a segment that is simultaneously affected by an incident and weather, the associated CAFs and SAFs are multiplied.

$\mathrm{CAF}^j_{\mathrm{inc}}$ and CAF^i_w are defined as the CAFs for type j incidents and type i weather events, respectively. For each segment and 15-min period, the joint CAF is computed by using Equation 5.33.

$$\mathrm{CAF} = \mathrm{CAF}^j_{\mathrm{inc}} \times \mathrm{CAF}^i_w \tag{5.33}$$

Similar calculations are considered for speed adjustments. SAF^j_{inc} and SAF^i_w are defined as the SAFs for type j incidents and type i weather events, respectively. Then for each segment and 15-min time period, the combined SAF is computed using Equation 5.34.

$$\mathrm{SAF} = \mathrm{SAF}^j_{\mathrm{inc}} \times \mathrm{SAF}^i_w \tag{5.34}$$

Figure 5.9 shows the CAF matrix generated by FSG, which is routed to FREEVAL-RL to adjust the capacity of the segments in every time period. Note the combined effect of CAF for segment 23 in time periods 12 through 14. The 0.62 values are computed as 0.93 (weather) × 0.67 (incident).

Figure 5.10 shows the SAF matrix generated by FSG, which is routed to FREEVAL-RL to adjust the free-flow speed of the segments in every time period.

Segment Number:	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
Segment Type:	B	OFR	OFR	B	ONR	ONR	R	OFR	B	W	B	W	B	W	B	ONR	ONR	B	OFR	OFR	B	ONR	B
t=1 (2:00 PM to 2:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=2 (2:15 PM to 2:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=3 (2:30 PM to 2:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=4 (2:45 PM to 3:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=5 (3:00 PM to 3:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=6 (3:15 PM to 3:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=7 (3:30 PM to 3:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=8 (3:45 PM to 4:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=9 (4:00 PM to 4:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=10 (4:15 PM to 4:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=11 (4:30 PM to 4:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=12 (4:45 PM to 5:00 PM)	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.62
t=13 (5:00 PM to 5:15 PM)	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.62
t=14 (5:15 PM to 5:30 PM)	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.62
t=15 (5:30 PM to 5:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.67
t=16 (5:45 PM to 6:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=17 (6:00 PM to 6:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=18 (6:15 PM to 6:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=19 (6:30 PM to 6:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=20 (6:45 PM to 7:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=21 (7:00 PM to 7:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=22 (7:15 PM to 7:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=23 (7:30 PM to 7:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=24 (7:45 PM to 8:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Figure 5.9. CAF table for I-40 EB case study detailed scenario 2117.

Segment Number:	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27
Segment Type:	B	OFR	OFR	B	ONR	ONR	R	OFR	B	W	B	W	B	W	B	ONR	ONR	B	OFR	OFR	B	ONR	B	OFR	B	ONR	ONR
t=1 (2:00 PM to 2:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=2 (2:15 PM to 2:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=3 (2:30 PM to 2:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=4 (2:45 PM to 3:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=5 (3:00 PM to 3:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=6 (3:15 PM to 3:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=7 (3:30 PM to 3:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=8 (3:45 PM to 4:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=9 (4:00 PM to 4:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=10 (4:15 PM to 4:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=11 (4:30 PM to 4:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=12 (4:45 PM to 5:00 PM)	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
t=13 (5:00 PM to 5:15 PM)	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
t=14 (5:15 PM to 5:30 PM)	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
t=15 (5:30 PM to 5:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=16 (5:45 PM to 6:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=17 (6:00 PM to 6:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=18 (6:15 PM to 6:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=19 (6:30 PM to 6:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=20 (6:45 PM to 7:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=21 (7:00 PM to 7:15 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=22 (7:15 PM to 7:30 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=23 (7:30 PM to 7:45 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
t=24 (7:45 PM to 8:00 PM)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Figure 5.10. SAF table for I-40 EB case study detailed scenario 2117.

Freeway Summary and Conclusions

The preceding sections of this chapter have presented the scenario generation process for evaluating travel time reliability on freeway facilities. In general, three factors affect travel time variability: traffic demand, weather, and incidents. The FSG converts these factors into an aggregated set of operational conditions on the facility, each with a predetermined probability. The mathematical performance model starts from the development of base, study period, and detailed scenarios. The latter are forwarded to the computational engine FREEVAL-RL for estimating analysis period facility travel times. Although full automation has yet to be accomplished, the process readily lends itself to automation.

The methodology combines the states of freeway operation to model weather events and incidents more realistically. Other factors that affect a facility's capacity or demand can be dealt with in a similar manner. The methodology transforms the probability distribution of base scenarios (representing operational conditions) into another space synchronized with the study period. The methodology also includes the determination of the required number of events that needs to be modeled in all study periods. Changes in event duration are accounted for by a change in the probability vector.

Urban Street Scenario Development

This section describes the scenario generation stage of the urban streets reliability methodology. Specifically, it describes the procedures used to create the scenarios that describe street and traffic conditions during the reliability reporting period. These procedures are as follows:

- Weather event procedure;
- Traffic demand variation procedure;
- Traffic incident procedure; and
- Scenario file generation procedure.

Weather Event Procedure

The weather event procedure is used to predict weather events (rain and snow) during the reliability reporting period. Also predicted is the time following each event that the pavement remains wet or covered by snow or ice. The presence of these conditions has been found to have an influence on running speed and intersection saturation flow rate. These effects are described later in this section.

The sequence of calculations in the weather event procedure is shown in Figure 5.11. The calculations proceed on a day-by-day basis in chronologic order. If a day is determined to have a weather event, its start time and duration are recorded for later use in the traffic incident procedure. Thereafter, each analysis period is evaluated in chronologic order for any given day with a weather event. If the analysis period is associated with a weather event, then the event type (i.e., rain or snow), precipitation rate (i.e., intensity), and pavement status (i.e., wet or snow-covered) are recorded for later use in the scenario file generation procedure.

The weather event procedure is based on the weather statistics in the following list. These statistics represent averages by month for 10 or more years. Default values are provided in the software implementation of the reliability methodology. They are described in Appendix H.

- Total normal precipitation;
- Total normal snowfall;
- Number of days with precipitation of 0.01 in. or more;
- Normal daily mean temperature; and
- Precipitation rate.

The weather event procedure consists of a series of calculation steps. A random number is used in several of the steps. All random numbers have a real value that is uniformly distributed from 0.0 to 1.0.

Step 1: Precipitation Prediction

The answer to the question of whether precipitation falls in a given day is based on a Monte Carlo method and an assumed binomial distribution of daily weather occurrence ($n = 1.0$, $x = 1.0$, $p = p(\text{precip})$). The probability of precipitation for any given day is computed by using Equation 5.35.

$$p(\text{precip})_m = \frac{\text{Ndp}_m}{\text{Nd}_m} \tag{5.35}$$

where

$p(\text{precip})_m$ = probability of precipitation in any given day of month m;

Ndp_m = number of days with precipitation of 0.01 in. or more in month m; and

Nd_m = total number of days in month m.

For each day considered, Equation 5.36 is checked to determine whether precipitation occurs.

$$\begin{aligned} &\text{No precipitation if } \text{Rp}_d \ge p(\text{precip}) \\ &\text{Precipitation if } \text{Rp}_d < p(\text{precip}) \end{aligned} \tag{5.36}$$

where Rp_d is equal to a random number for precipitation for day d.

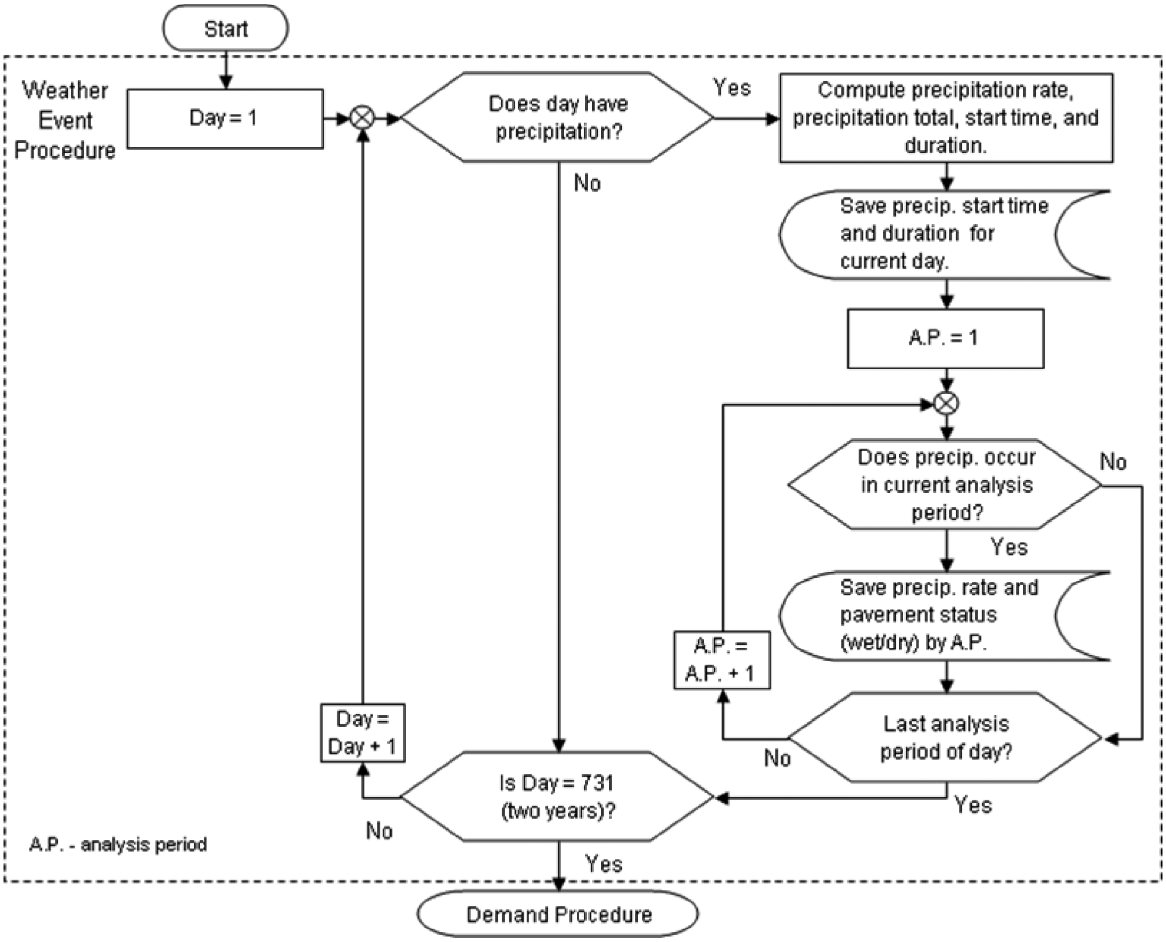


Figure 5.11. Weather event procedure.

Step 2: Precipitation Type

If precipitation occurs, then Equation 5.37 is used to estimate the average temperature during the weather event for the subject day.

$$T_{d,m} = \text{normal}^{-1}\left(p = \text{Rg}_d, \mu = \overline{T}_m, \sigma = s_T\right) \tag{5.37}$$

where

$T_{d,m}$ = average temperature for day d of month m, °F;
Rg_d = random number for temperature for day d;
$\overline{T}_m$ = normal daily mean temperature in month m, °F;
s_T = standard deviation of daily mean temperature in a month (= 5.0), °F; and
$\text{normal}^{-1}(p, \mu, \sigma)$ = value associated with probability p for cumulative normal distribution with mean μ and standard deviation σ.

The average temperature for the day is used to determine whether the precipitation is in the form of rain or snow. The temperature variation during the day can influence this determination. However, modeling this influence is rationalized to add more analytic sophistication than is justified for the reliability evaluation. Therefore, for each day considered in the reliability reporting period, Equation 5.38 is used to determine whether the precipitation that day is in the form of rain or snow.

$$\begin{aligned} &\text{Rain if } T_{d,m} \geq 32°\text{F} \\ &\text{Snow if } T_{d,m} < 32°\text{F} \end{aligned} \tag{5.38}$$

The normal daily mean temperature is obtained from the National Climatic Data Center's Comparative Climatic Data (NCDC 2011a). This statistic is tabulated by month of year for 284 U.S. cities and territories. It represents the average of the daily mean temperatures in a given month.

The standard deviation of the daily mean temperature s_T represents the variability of the daily mean temperature for a given month. This statistic was computed for seven U.S. cities. The cities represent all combinations of north/south and east/middle/west regions of the country. The daily mean temperature data were obtained from the National Climatic Data Center (NCDC

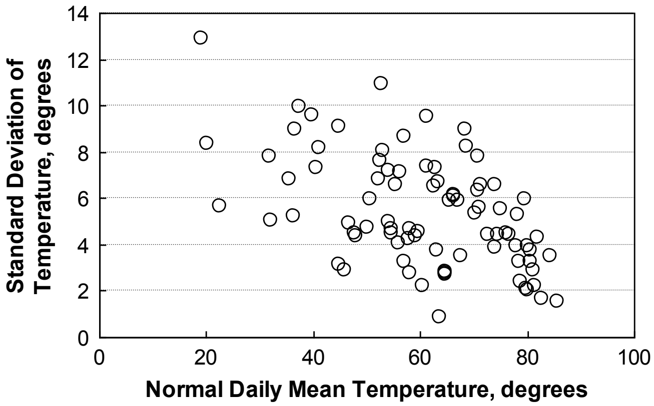


Figure 5.12. Standard deviation of daily mean temperature (°F).

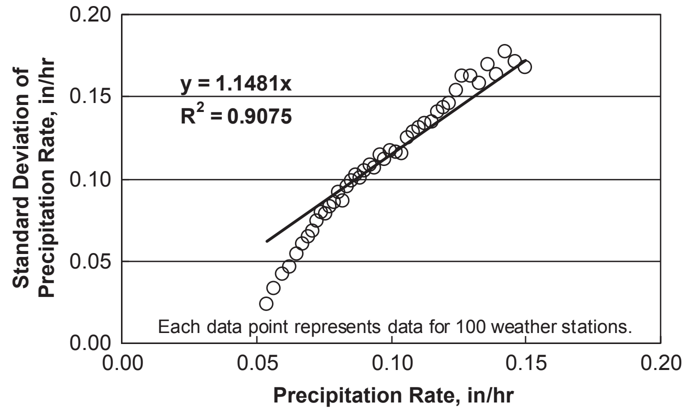


Figure 5.13. Standard deviation of precipitation rate.

2011c). The standard deviation for the seven cities is shown in Figure 5.12 as a function of the normal daily mean temperature. Twelve observations were recorded for each city.

The general trend in the data suggests that the standard deviation decreases slightly with an increase in the normal daily mean temperature ($R^2 = 0.28$). However, the standard deviation did not vary with temperature for cities on the West Coast. On the basis of this examination, a constant value of 5.0°F is recommended for s_T for the reliability evaluation.

Step 3: Rain Intensity

When evaluated on an hourly basis, the rainfall rate (i.e., intensity) can be highly variable. The gamma distribution has the ability to replicate nonnegative random variants that are highly variable. Equation 5.39 is used to estimate the rainfall rate during a rain event.

$$\mathrm{rr}_{d,m} = \mathrm{gamma}^{-1}\left(p = \mathrm{Rr}_d, \mu = \overline{\mathrm{rr}}_m, \sigma = s_{\mathrm{rr},m}\right) \tag{5.39}$$

where

- $\mathrm{rr}_{d,m}$ = rainfall rate for the rain event occurring on day d of month m, in./h;
- Rr_d = random number for rainfall rate for day d;
- $\overline{\mathrm{rr}}_m$ = precipitation rate in month m, in./h;
- $s_{\mathrm{rr},m}$ = standard deviation of precipitation rate in month m (= 1.0 $\overline{\mathrm{rr}}_m$), in./h; and
- $\mathrm{gamma}^{-1}(p, \mu, \sigma)$ = value associated with probability p for cumulative gamma distribution with mean μ and standard deviation σ.

The average precipitation rate (and its standard deviation) is based on time periods when precipitation is falling. Thus, the average precipitation rate represents an average for all hours in which precipitation is falling (and excludes any hours in which precipitation is not falling).

Binned hourly precipitation data were obtained from the National Climatic Data Center to examine the standard deviation of precipitation rate (NCDC 2011b). Data were obtained for 5,900 weather stations collectively representing all 50 states. An examination of the data indicates that the standard deviation of the precipitation rate is about equal to the average precipitation rate. The general trend in this relationship is shown in Figure 5.13.

For the reliability evaluation, the standard deviation is conservatively assumed to equal the average precipitation rate (i.e., $s_{\mathrm{rr},m} = 1.0\overline{\mathrm{rr}}_m$). This approach excludes the very rare intense storm event and increases the intensity slightly of those events that do occur.

Equation 5.40 is used to estimate the total amount of rainfall for a rain event. This analysis assumes that each day with precipitation has one rain event.

$$\mathrm{tr}_{d,m} = \mathrm{gamma}^{-1}\left(p = \mathrm{Rt}_d, \mu = \overline{\mathrm{tr}}_m, \sigma = s_{\mathrm{tr},m}\right) \tag{5.40}$$

with

$$\overline{\mathrm{tr}}_m = \frac{\mathrm{tp}_m}{\mathrm{Ndp}_m} \tag{5.41}$$

$$s_{\mathrm{tr},m} = \text{Smaller of } \left(2.5\,\overline{\mathrm{tr}}_m, 0.65\right) \tag{5.42}$$

where

- $\mathrm{tr}_{d,m}$ = total rainfall for the rain event occurring on day d of month m, in./event;
- Rt_d = random number for rainfall total for day d (= Rr_d);
- $\overline{\mathrm{tr}}_m$ = average total rainfall per event in month m, in./event;
- $s_{\mathrm{tr},m}$ = standard deviation of total rainfall in month m, in./event; and
- tp_m = total normal precipitation for month m, in.

Total rainfall for a rain event represents the product of the rainfall rate and the rain event duration. Thus, the total rainfall amount is highly correlated with the rainfall rate. For the reliability evaluation, total rainfall is assumed to be perfectly correlated with rainfall rate such that they share the same random number. This approach may result in slightly less variability in the estimated total rainfall; however, it precludes the occasional calculation of unrealistically long or short rain events.

The standard deviation of total rainfall combines the variability of rainfall rate and rainfall duration. As a result, its value is defined partly by the manner in which the rainfall rate is defined (i.e., measured) and the rain event is modeled. The approach used to calibrate the standard deviation of total rainfall is to compare the resulting estimates of rainfall rate and rain duration for a range of values. The calculation of rainfall duration is described in the next step.

Harwood et al. (1988) examined rainfall rate and duration data for 99 weather stations in 22 metropolitan areas throughout the United States. The relationship they found between the two variables is shown in Figure 5.14 using the solid trend line. Equations 5.39 through 5.43 were used to compute the rainfall rate and duration for various percentile events (i.e., the random number for Equations 5.39 and 5.40 was set to a common percentile value). The percentile values included 0.01, 0.1, 0.5, 0.9, and 0.99.

The relationship between the computed rainfall rate and duration is shown in Figure 5.14. The circles represent the data points corresponding to various percentile values. The standard deviation relationship shown in Equation 5.42 was derived to provide the best fit between the data points and the trend line representing the findings by Harwood et al. (1988).

Step 4: Rainfall Duration

Equation 5.43 is used to estimate the rainfall duration for a rain event.

$$\mathrm{dr}_{d,m} = \frac{\mathrm{tr}_{d,m}}{\mathrm{rr}_{d,m}} \tag{5.43}$$

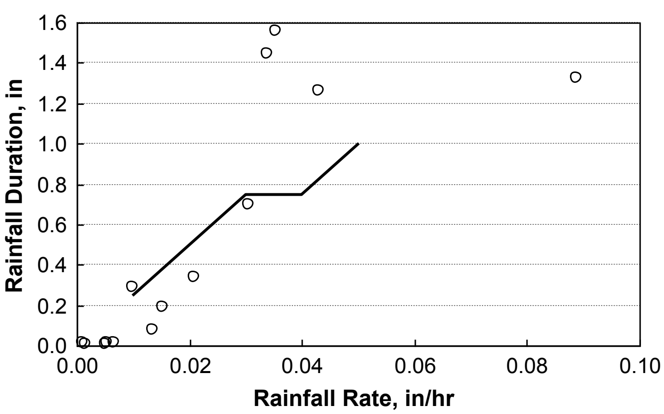


Figure 5.14. Relationship between rainfall duration and rainfall rate.

where $\mathrm{dr}_{d,m}$ is rainfall duration for the rain event occurring on day *d* of month *m,* h/event.

The duration computed in Equation 5.43 is used in a subsequent step to determine whether an analysis period is associated with a rain event. To simplify the analytics in this subsequent step, it is assumed that no rain event extends beyond midnight. To ensure this outcome, the duration computed in Equation 5.43 is compared with the time duration between the start of the study period and midnight. The rainfall duration is then set to equal the smaller of the two values.

Step 5: Start Time of Weather Event

For the reliability methodology, the hour of day that the rain event starts is determined randomly. The start hour is computed using Equation 5.44:

$$\mathrm{ts}_{d,m} = (24 - \mathrm{dr}_{d,m})\, R_{s,d} \tag{5.44}$$

where $\mathrm{ts}_{d,m}$ is the start of rain event on day *d* of month *m,* in hours; and $R_{s,d}$ is the random number for rain event start time for day *d.*

The start time from Equation 5.44 is rounded to the nearest hour for 1-hour analysis periods or to the nearest quarter hour for 15-min analysis periods.

Step 6: Wet-Pavement Duration

Following a rain event, the pavement remains wet for some length of time. The presence of wet pavement can influence road safety by reducing surface-tire friction. Research by Harwood et al. (1988) indicates that wet-pavement time can be computed using Equation 5.45:

$$\mathrm{dw}_{d,m} = \mathrm{dr}_{d,m} + \mathrm{do}_{d,m} + \mathrm{dd}_{d,m} \tag{5.45}$$

with

$$\mathrm{dd}_{d,m} = 0.888\ \exp(-0.0070\, T_{d,m}) + 0.19 I_{\mathrm{night}} \tag{5.46}$$

where

- $\mathrm{dw}_{d,m}$ = duration of wet pavement for rain event occurring on day *d* of month *m,* h/event;
- $\mathrm{do}_{d,m}$ = duration of pavement runoff for rain event occurring on day *d* of month *m* (= 0.083), h/event;
- I_{night} = indicator variable for day/night (= 0.0 if rain starts between 6:00 a.m. and 6:00 p.m., 1.0 otherwise); and
- $\mathrm{dd}_{d,m}$ = duration of drying time for rain event occurring on day *d* of month *m,* h/event.

The duration computed with Equation 5.45 is used in a subsequent step to determine whether an analysis period is associated with wet-pavement conditions. To simplify the analytics in that subsequent step, it is assumed that no rain event extends beyond midnight. To ensure this outcome, the duration computed from Equation 5.45 is compared with the time duration between the start of the rain event and midnight. The wet-pavement duration is then set to equal the smaller of the two values.

Pavement runoff duration represents the time period after rainfall ends when rainwater is running off the pavement. The runoff duration is influenced by rainfall intensity, pavement surface texture, and the pavement cross slope. Research by Harwood et al. (1988) states that runoff duration is usually less than 10 min. They also indicate that 5 min can be considered a representative rainfall runoff duration.

Harwood et al. (1988) also investigated the duration of pavement drying. They conducted pavement drying tests in a laboratory and confirmed their findings in the field. They found that the average drying period lasted 31.6 min, which was consistent with two other studies (Harwood et al. 1988). The drying time was found to vary with relative humidity, day versus night, cloudy versus clear, wind speed, and pavement type. They described a categorical model for estimating drying duration. The model indicated that drying time increased 11.6 min during nighttime hours. The relationship between temperature and drying duration obtained from this model is shown in Figure 5.15 using a thick trend line. The trend line is stair-stepped because of the categorical way the researchers chose to present their model.

The best-fit regression trend line is shown as a thin line in Figure 5.15. The equation for this line is also shown in the figure (and included in Equation 5.45). It is extrapolated to temperatures as low as 10°F. The trend is plausible and is recommended for the reliability evaluation until additional research is conducted to develop a more accurate relationship.

Step 7: Snow Intensity and Duration

The snowfall rate (i.e., intensity) and duration are computed using the calculation sequence in Steps 3 through 6. The

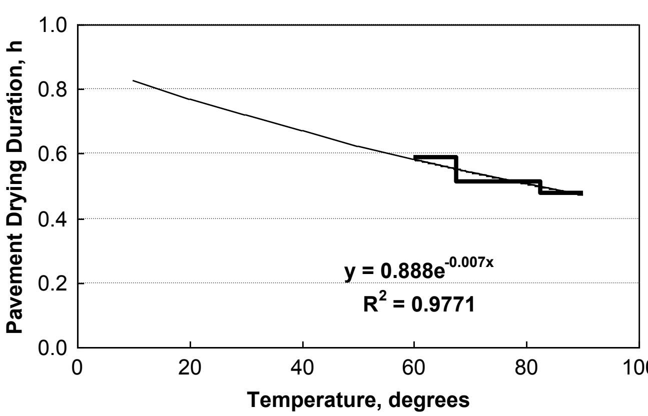


Figure 5.15. Relationship between pavement drying duration and temperature (°F).

equations are the same. The average snowfall rate and average snow total per event are computed by multiplying the average precipitation rate and average total rainfall per event, respectively, by the ratio of snow depth to rain depth. This ratio is estimated at 10 in./in. on the basis of an analysis of weather data reported by the National Climatic Data Center (2011a).

In Step 6, the duration of pavement runoff is defined differently when applied to snow events. Specifically, it is defined as the time after the snow stops falling that snow pack (or ice) covers the pavement. After this time period elapses, the pavement is exposed, and drying begins. This time is likely a function of traffic volume, snow depth, and agency snow removal capabilities. An overall average value of 30 min is estimated for this variable. Additional research is needed to quantify this value for typical conditions.

Step 8: Identifying Analysis Period Weather

Steps 1 through 7 are repeated for each day of a 2-year period, starting with the first day of the reliability reporting period. This 2-year record of weather events is used in the traffic incident procedure to estimate the weather-related incident frequency.

The days that have weather events are subsequently examined to determine whether the event occurs during the study period. Specifically, each analysis period is examined to determine whether it is associated with a weather event. If the pavement is wet during an analysis period, then the precipitation type (i.e., rain or snow) is recorded for that period. If precipitation is falling, then the precipitation rate is also recorded.

The durations of precipitation and wet pavement from Equations 5.43 and 5.45, respectively, are rounded to the nearest hour for 1-hour analysis periods, or to the nearest quarter hour for 15-min analysis periods. This rounding is performed to ensure the most representative match between event duration and analysis period start and end times. This approach causes events that are shorter than one-half of the analysis period duration to be ignored (i.e., they are not recognized in the scenario generation process). The use of a 15-min analysis period duration minimizes the number of events that are ignored, relative to a 1-hour analysis period.

Traffic Demand Variation Procedure

The traffic demand variation procedure is used to identify the appropriate traffic demand adjustment factors for each analysis period in the reliability reporting period. One set of factors accounts for systematic volume variation by hour of day, day of week, and month of year. Default values for these factors are provided in the software implementation of the reliability methodology. They are described in Appendix H.

The sequence of calculations in the traffic demand variation procedure is shown in Figure 5.16. The calculations proceed on a day-by-day and hour-by-hour basis in chronologic

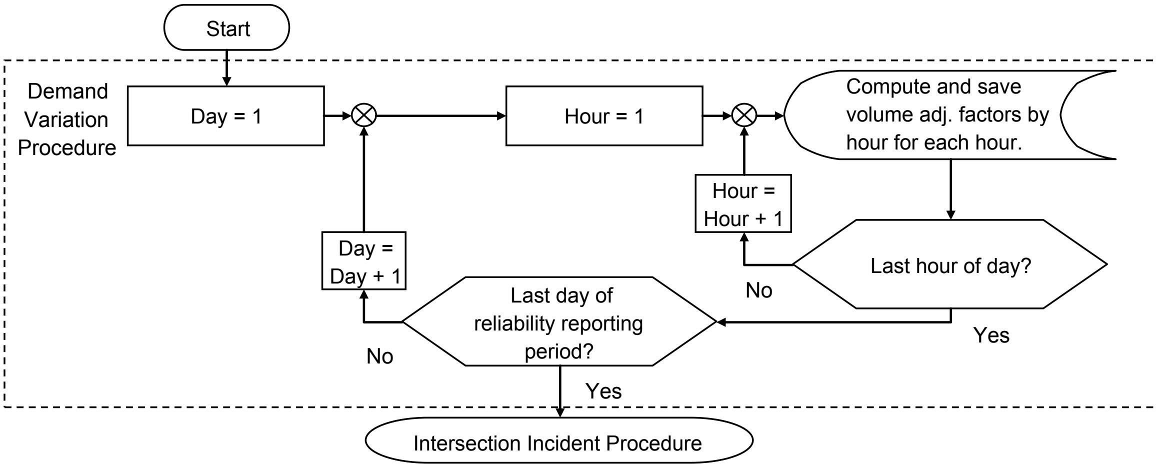


Figure 5.16. Traffic demand variation procedure.

order. Within a given day, the procedure considers only those hours that occur during the study period. The factors identified in this procedure are subsequently used in the scenario file generation procedure to compute the demand volume for the subject urban street facility.

A random variation adjustment factor is also available and can be included, if desired, by the analyst. It accounts for the random variation in volume that occurs among 15-min time periods. This factor is described in more detail in the Scenario File Generation Procedure subsection later in this chapter.

The procedure includes two adjustment factors to account for a reduction in traffic demand during inclement weather. One factor addresses the demand change when it is raining. The second factor addresses demand change when it is snowing. Maki (1999) examined traffic volume on an urban street in Minnesota and found that traffic volumes were 15% to 30% lower when it was snowing. She rationalized that motorists altered the start time of their commute, or just stayed home, to avoid the bad weather. Research on freeway traffic volume indicates a similar reduction and sensitivity to snowfall rate (Hanbali and Kuemmel 1993; Ibrahim and Hall 1994); however, whether this trend extends to urban street traffic is not clear. Ibrahim and Hall also found that light rain had no effect on freeway volume, while a heavy rain reduced volume by 10% to 20%. Given these findings, a factor of 1.0 is recommended for rain events and a factor of 0.80 is recommended for snow events.

This procedure does not predict traffic diversion related to the presence of work zones or special events. The accommodation of this diversion in a reliability evaluation is discussed in Chapter 4, in the work zones and special events subsection.

Leaving the date of the traffic count blank in the base input file implies that the volumes in the file are based on planning estimates of volume during the average day of week and month of year. In this situation, the adjustment factors for day of week and month of year are set to a value of 1.0. A similar determination is made if no date is entered for the traffic counts in the alternative input files.

The volumes entered in an input file are assumed to reflect the directional distribution of traffic during the specified study period. If this distribution varies significantly during certain periods of the day (e.g., a.m. peak or p.m. peak), then each unique period should be the focus of a separate reliability evaluation. When multiple SP evaluations are undertaken for a common facility, the set of analysis period averages (APA) for each evaluation can be merged to evaluate the overall reliability. This merging process is done manually, using the cut and paste functions of the spreadsheet software that implements the reliability methodology.

Traffic Incident Procedure

The traffic incident procedure is used to predict incident date, time, and duration. It also determines incident event type (i.e., crash or noncrash), severity level, and location on the facility. Location is defined by the specific intersection or segment on which the incident occurs and whether the incident occurs on the shoulder, one lane, or multiple lanes. The procedure uses weather event and traffic demand variation information from the previous procedures in the incident prediction process.

The sequence of calculations in the traffic incident procedure is shown in Figure 5.17. The sequence shown is applicable to incidents occurring at signalized intersections. A similar sequence is followed for incidents occurring at locations along the urban street between the signalized intersections (i.e., mid-signal segments).

The traffic incident procedure consists of a set of calculation steps. The calculations associated with each step are described in this subsection. A random number is used in several of the steps. All random numbers have a real value that is uniformly distributed from 0.0 to 1.0.

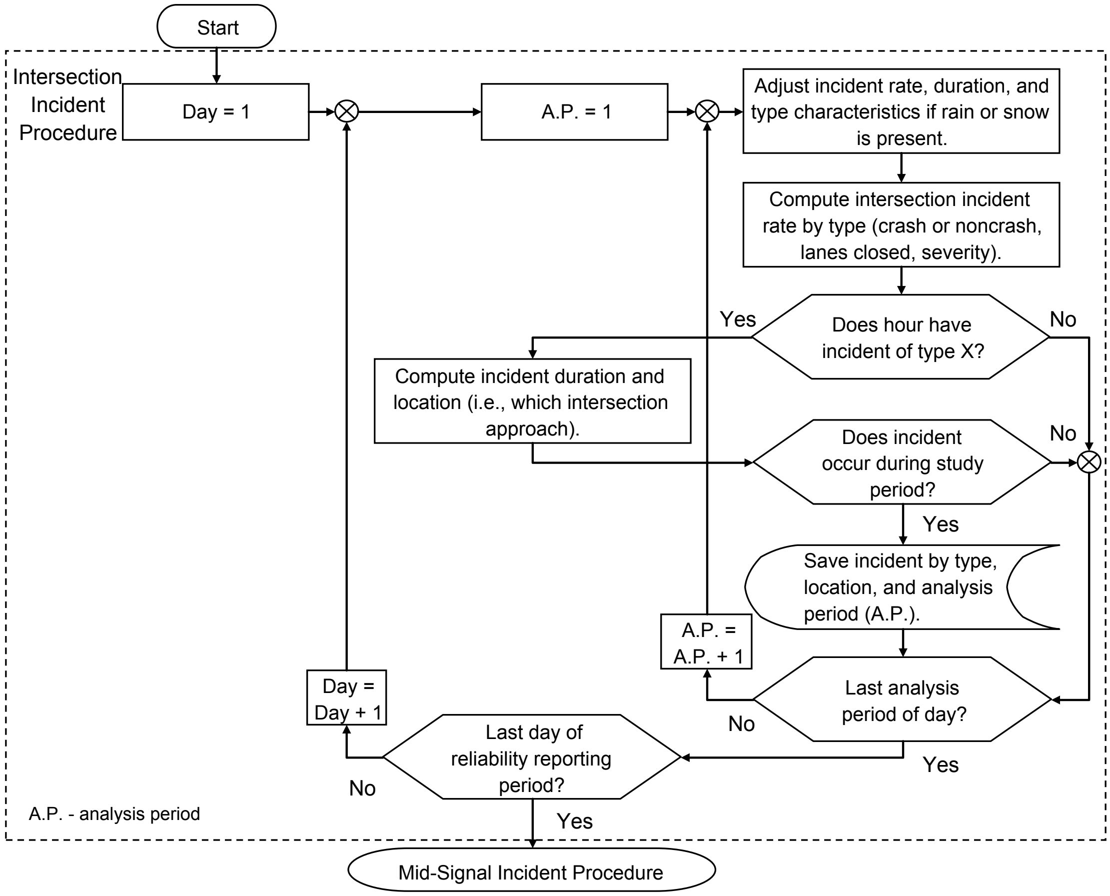


Figure 5.17. Traffic incident procedure for intersection incidents.

Step 1: Compute the Equivalent Crash Frequency for Weather

A review of the safety literature indicates that crash frequency increases when the road is wet, covered by snow, or covered by ice (Maze et al. 2005). The effect of weather on crash frequency is incorporated in the reliability methodology by converting the input crash frequency data into an equivalent crash frequency for each type of weather condition. The equivalent crash frequency for dry pavement conditions is defined using Equation 5.47:

$$Fc_{str(i),dry} = Fc_{str(i)} \frac{8760\ Ny}{Nh_{dry} + CFAF_{rf}\ Nh_{rf} + CFAF_{wp}\ Nh_{wp} + CFAF_{sf}\ Nh_{sf} + CFAF_{sp}\ Nh_{sp}} \quad (5.47)$$

where

$Fc_{str(i),\,dry}$ = equivalent crash frequency when every day is dry for street location i of type str (str = int: intersection, seg: segment), crashes/year;

$Fc_{str(i)}$ = expected crash frequency for street location i of type str, crashes/year;

Ny = total number of years, years;

Nh_{dry} = total number of hours in Ny years with dry conditions, h;

Nh_{rf} = total number of hours in Ny years with rainfall conditions, h;

Nh_{wp} = total number of hours in Ny years with wet pavement and not raining, h;

Nh_{sf} = total number of hours in Ny years with snowfall conditions, h;

Nh_{sp} = total number of hours in Ny years with snow or ice on pavement and not snowing, h;

$CFAF_{rf}$ = crash frequency adjustment factor for rainfall (= 2.0);

$CFAF_{wp}$ = crash frequency adjustment factor for wet pavement (not raining) (= 3.0);

$CFAF_{sf}$ = crash frequency adjustment factor for snowfall (= 1.5); and

$CFAF_{sp}$ = crash frequency adjustment factor for snow or ice on pavement (not snowing) (= 2.75).

The equivalent crash frequency for nondry conditions is computed using Equation 5.48. The crash frequency adjustment factor (CFAF) for dry weather, $CFAF_{str(i),dry}$, is 1.0.

$$Fc_{str(i),\,wea} = Fc_{str(i),\,dry}\ CFAF_{wea} \quad (5.48)$$

where $Fc_{str(i),\,wea}$ is the equivalent crash frequency when every day has weather condition wea (wea = dry: no precipitation and dry pavement, rt: rainfall, wp: wet pavement but not raining, sf: snowfall, sp: snow or ice on pavement but not snowing) for street location *i* of type str (str = int: intersection, seg: segment), crashes/year.

A 2-year weather history is created by the weather event procedure and is used to compute the total number of hours for each weather condition in the vicinity of the subject facility. A 2-year history is used to reduce the random variability in weather event duration.

This step is separately applied to each intersection and segment on the facility. When applied to intersections, the expected crash frequency Fc is the value input for the subject intersection. It is the value input for the subject segment when applied to segments.

The CFAF represents the ratio of hourly crash frequency during the weather event divided by the hourly crash rate during clear, dry hours. It is computed using one or more years of historic weather data and crash data for the region in which the subject facility is located. The CFAF for a specific weather condition is computed by identifying (1) the number of hours for which the weather condition exists for the year, and (2) the count of crashes during those hours. An hourly crash frequency for the weather condition (fc_{wea}) is computed by dividing the crash count by the number of hours. Using a similar technique, the hourly crash frequency is computed for dry pavement hours (fc_{dry}). The CFAF for the weather condition is computed as the ratio of these two frequencies (i.e., $CFAF_{wea} = fc_{wea}/fc_{dry}$).

The CFAF includes consideration of the effect of the weather event on traffic volume (i.e., volume may be reduced because of bad weather) and on crash risk (i.e., wet pavement may increase the potential for a crash). For example, if rainfall is envisioned to increase crash risk by 200% and to decrease traffic volume by 10%, then the CFAF for rainfall is 2.70 (= 3.0 × 0.9).

The literature was reviewed to determine if default CFAF values could be derived. The sources found indicate little agreement on how to quantify the effect of weather on safety. Some researchers based their evaluation on crash counts during various weather conditions, while others based their evaluation on crash rates. Still other researchers compared crash data (count or rate) for days with a weather event with that for days with dry conditions (thus assuming that the weather event affected safety for a 24-hour period, even if the weather event lasted only a few minutes). Table 5.28 summarizes the findings from the literature (Andrey et al. 2001; Bijleveld and Churchill 2009; SWOV 2009; Brodsky and Hakkert 1988).

Three rows in the lower half of Table 5.28 summarize the CFAF values derived from data for arterial streets in three cities. These values are derived from 11,308; 971; and 135 crashes reported for Louisville, San Mateo, and Portland, respectively. The last row of the table shows the recommended CFAF values. These values are intended to represent the trends shown in Table 5.28.

Step 2: Establish the CFAFs for Work Zones or Special Events

If the analysis period occurs during a work zone or special event, then the CFAF variable for segments $CFAF_{str}$ and the CFAF variable for intersections $CFAF_{int}$ are set to the values

Table 5.28. Crash Frequency Adjustment Factors

Source	Weather Condition			
	Raining	Snowing	Clear with Wet Pavement	Clear with Snow or Ice on Pavement
Andrey et al. (2001)	1.75	N/A	N/A	N/A
Andrey et al. citing O'Leary (1978)	N/A	2.5	N/A	N/A
Andrey et al. citing Bertness (1980)	>2.0	N/A	N/A	N/A
Andrey et al. citing Robinson (1965)	1.3	N/A	N/A	N/A
SWOV (2009)	2.0	N/A	N/A	N/A
Streets in Louisville, Kentucky[a]	1.83	0.58	4.04	2.61
Streets in San Mateo, California[a]	1.95	N/A	1.99	N/A
Streets in Portland, Oregon[a]	2.01	N/A	4.19	N/A
Recommended CFAF value	**2.00**	**1.50**	**3.00**	**2.75**

Note: N/A = not applicable, data not available.
[a] CFAF values from these sources were derived from data for arterial streets: 11,308 crashes reported for Louisville; 971 crashes reported for San Mateo; and 135 crashes reported for Portland.

provided by the analyst. Otherwise, $CFAF_{str}$ and $CFAF_{int}$ equal 1.0. This step is repeated for each day of the reliability reporting period.

Step 3: Determine Whether an Incident Occurs

During this step, each of the 24 hours in the subject day is examined to determine if an incident occurs. The analysis separately considers each street location (i.e., intersection and segment). At each street location, each of the following 12 incident types is separately addressed. Each of these types is separately considered for each hour of the day. (Whether the hour coincides with an analysis period is determined in a subsequent step.)

- Crash, one lane blocked, fatal or injury;
- Crash, two or more lanes blocked, fatal or injury;
- Crash, shoulder location, fatal or injury;
- Crash, one lane blocked, property damage only;
- Crash, two or more lanes blocked, property damage only;
- Crash, shoulder location, property damage only;
- Noncrash, one lane blocked, breakdown;
- Noncrash, two or more lanes blocked, breakdown;
- Noncrash, shoulder location, breakdown;
- Noncrash, one lane blocked, other;
- Noncrash, two or more lanes blocked, other; and
- Noncrash, shoulder location, other.

Initially, the weather event data are checked to determine whether the subject day and hour are associated with rainfall, wet pavement and not raining, snowfall, or snow or ice on pavement and not snowing. For a given day, street location, and hour of day, the average incident frequency is computed using Equation 5.49, which is based on the weather present at that hour and day.

$$Fi_{str(i),\,wea(h,\,d)} = CFAF_{str}\,\frac{Fc_{str(i),\,wea}}{pc_{str,\,wea}} \tag{5.49}$$

where

$Fi_{str(i),wea(h,d)}$ = expected incident frequency for street location i of type str and weather condition wea(h,d) during hour h and day d, incidents/year;

$CFAF_{str}$ = crash frequency adjustment factor for street location type str; and

$pc_{str,wea}$ = proportion of incidents that are crashes for street location type str and weather condition wea (= 0.358 for segments and 0.310 for intersections).

Dowling et al. (2011) collected incident data for several arterial streets in California and Oregon. Data were collected for five California streets totaling 86.5 miles and for two Oregon streets totaling 22 miles. There are 2,207 incidents included in the combined database. The proportion of these incidents that are crashes was computed as 0.358 for segments and 0.310 for intersections. These values reflect an average for all weather conditions. Additional research is needed to quantify these variables by weather condition.

The incident frequency is converted to an hourly frequency that is sensitive to traffic demand variation by hour of day, day of week, and month of year. The converted frequency is computed using Equation 5.50.

$$Fi_{str(i),\,wea(h,\,d),\,h,\,d} = \frac{Fi_{str(i),\,wea(h,\,d)}}{8{,}760}\left(24\ f_{hod,\,h,\,d}\right) f_{dow,\,d}\ f_{moy,\,d} \tag{5.50}$$

where

$Fi_{str(i),wea(h,d),h,d}$ = expected hourly incident frequency for street location i of type str and weather condition wea(h,d) during hour h and day d, incidents/h;

$f_{hod,\ h,d}$ = hour-of-day adjustment factor based on hour h and day d;

$f_{dow,d}$ = day-of-week adjustment factor based on day d; and

$f_{moy,d}$ = month-of-year adjustment factor based on day d.

The hour-of-day adjustment factor includes a day subscript because its values vary depending on whether the day occurs during a weekday or weekend. The day subscript for the day-of-week factor is used to determine which of the seven weekdays is associated with the subject day. Similarly, this subscript is used to determine which of the 12 months is associated with the subject day for the month-of-year factor. Default values for these adjustment factors are described in Appendix H.

Incidents for a given day, street location, incident type, and hour of day are assumed to follow a Poisson distribution. For any given combination of conditions, the probability of more than one incident is negligible, which simplifies the mathematics such that the question of whether an incident occurs is reduced to whether there are zero incidents or one incident. Equation 5.51 is used to compute the probability of no incidents occurring. Default values for the proportion of incidents are listed in Appendix H.

$$p0_{str(i),\,wea(h,\,d),\,con,\,lan,\,sev,\,h,\,d} = \exp\begin{pmatrix} -fi_{str(i),\,wea(h,\,d),\,h,\,d} \\ \times\ pi_{str(i),\,wea(h,d),\,con,\,lan,\,sev} \end{pmatrix} \tag{5.51}$$

where

$p0_{str(i),wea(h,d),\ con,\ lan,\ sev,h,d}$ = probability of no incident for street location i of type str, weather condition wea(h,d) during hour h and day d, event type con (con = cr: crash, nc: noncrash), lane location lan (lan = 1*L*: one lane, 2*L*: two or more lanes, sh: shoulder), and severity sev (sev = pdo: property damage only, fi: fatal or injury, bkd: breakdown, oth: other); and

$pi_{\text{str, wea}(h,d)\text{, con, lan, sev}}$ = proportion of incidents for street location type str, weather condition wea(*h,d*) during hour *h* and day *d,* event type con, lane location lan, and severity sev.

The following rule (Equation 5.52) is checked to determine whether the incident of a specific type occurs.

No incident if

$$Ri_{\text{str}(i),\text{wea}(h,d),\text{con,lan,sev},h,d} \le p0_{\text{str}(i),\text{wea}(h,d),\text{con,lan,sev}} \tag{5.52}$$

Incident if

$$Ri_{\text{str}(i),\text{wea}(h,d),\text{con,lan,sev},h,d} > p0_{\text{str}(i),\text{wea}(h,d),\text{con,lan,sev}}$$

where $Ri_{\text{str}(i),\text{ wea}(h,d),\text{ con, lan, sev},h,d}$ is a random number for incident for street location *i* of type str, weather condition wea(*h,d*) during hour *h* and day *d,* event type con, lane location lan, and severity sev.

Step 4: Determine Incident Duration

If the result of Step 3 indicates that an incident occurs for a given day, street location, incident type, and hour of day, then the calculations in this step are used to determine the incident duration. Each hour of the day is separately considered in this step.

Incident duration includes the incident detection time, response time, and clearance time. Research indicates that these values can vary by weather condition, event type, lane location, and severity (List et al. 2008; Dowling et al. 2011; Raub and Schofer 1997). Default values for average incident duration are provided in Appendix H.

The data indicate that incident duration can be highly variable (List et al. 2008; Raub and Schofer 1997). The gamma distribution has the ability to replicate nonnegative random variates that are highly variable. Equation 5.53 is used to estimate the incident duration for a given incident.

$$di_{\text{str}(i),\text{wea}(h,d),\text{con,lan,sev},h,d} = \text{gamma}^{-1}\begin{pmatrix} p = Rd_{\text{str}(i),\text{con,lan,sev},h,d}, \\ \mu = \overline{di}_{\text{str,wea}(h,d),\text{con,lan,sev}}, \\ \sigma = s_{\text{str,wea}(h,d),\text{con,lan,sev}} \end{pmatrix} \tag{5.53}$$

where

$di_{\text{str}(i),\text{ wea}(h,d),\text{ con, lan, sev},h,d}$ = incident duration for street location *i* of type str, weather condition wea(*h,d*) during hour *h* and day *d,* event type con, lane location lan, and severity sev, *h*;

$Rd_{\text{str}(i),\text{ con, lan, sev},h,d}$ = random number for incident duration for street location *i* of type str for hour *h* and day *d,* event type con, lane location lan, and severity sev;

$\overline{di}_{\text{str, wea}(h,d)\text{,con, lan, sev}}$ = average incident duration for street location type str, weather condition wea(*h,d*) during hour *h* and day *d,* event type con, lane location lan, and severity sev, *h*;

$s_{\text{str, wea}(h,d)\text{, con, lan, sev}}$ = standard deviation of incident duration for street location type str, weather condition wea(*h,d*) during hour *h* and day *d,* event type con, lane location lan, and severity sev ($= 0.8\ \overline{di}_{\text{str, wea}(h,d)\text{,con, lan, sev}}$), *h*; and

$\text{gamma}^{-1}(p, \mu, \sigma)$ = value associated with probability *p* for cumulative gamma distribution with mean μ and standard deviation σ.

The duration computed with Equation 5.53 is used in a subsequent step to determine whether an analysis period is associated with an incident. To simplify the analytics in that subsequent step, it is assumed that no incident extends beyond midnight. To ensure this outcome, the duration computed from Equation 5.53 is compared with the time duration between the start of the study period and midnight. The incident duration is then set to equal the smaller of the two values.

The incident duration data were examined to determine an appropriate standard deviation of incident duration (Raub and Schofer 1997). This examination indicated that the standard deviation was correlated with the average incident duration. The standard deviation of crash-related incidents was equal to 60% of the average duration. The standard deviation of the non-crash-related incidents was equal to 110% of the average duration.

The data reported also demonstrated a strong correlation between standard deviation and average incident duration (List et al. 2008). This relationship is shown in Figure 5.18. The standard deviation is shown to be about 87% of the average duration. Each data point represents a different combination of event type, lane location, and severity. The difference between crash and noncrash incidents noted in the Raub and Schofer data was not found in the List et al. (2008) data. On the basis of this finding about the Raub and Schofer and List et al. (2008) data, the standard deviation for all incident types is estimated to equal 0.8 times the average incident duration.

Step 5: Determine Incident Location

If the result of Step 3 indicates that an incident occurs for a given day, street location, incident type, and hour of day, then

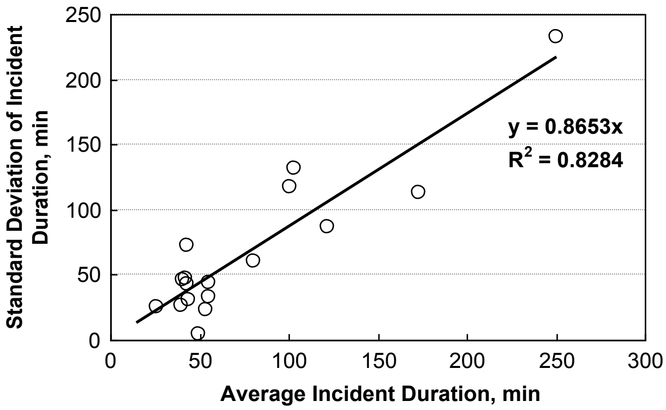


Figure 5.18. Standard deviation of incident duration.

Step 5 is used to determine the incident location. For intersections, the location is determined to be one of the intersection legs. For segments, the location is determined to be one of the two travel directions. The location algorithm is volume-based so that the correct location determinations are made when addressing three-leg intersections or one-way streets. Each hour of the day is considered separately in this step.

Intersection Location

When a specific intersection is associated with an incident, the location of the incident is based on consideration of each intersection leg volume lv. This volume represents the sum of all movements entering the intersection on the approach lanes plus those movements exiting the intersection on the adjacent departure lanes. In the field, this volume would be measured by establishing a reference line from outside curb to outside curb on the subject leg (near the crosswalk) and counting all vehicles that cross the line, regardless of travel direction.

The leg volumes are then summed, starting with the leg associated with National Electrical Manufacturers Association (NEMA) Phase 2, to produce a cumulative volume by leg. These volumes are then converted to a proportion by dividing by the sum of the leg volumes. The calculation of these proportions is described by Equations 5.54 and 5.55. One set of proportions is determined for the base input file and for each work zone and special event input file.

$$\begin{aligned}
pv_{\text{int}(i),2} &= lv_{\text{int}(i),2} / (2\, tv_{\text{int}(i)}) \\
pv_{\text{int}(i),6} &= pv_{\text{int}(i),4} + lv_{\text{int}(i),6} / (2\, tv_{\text{int}(i)}) \\
pv_{\text{int}(i),4} &= pv_{\text{int}(i),2} + lv_{\text{int}(i),4} / (2\, tv_{\text{int}(i)}) \\
pv_{\text{int}(i),8} &= 1.0
\end{aligned} \tag{5.54}$$

with

$$tv_{\text{int}(i)} = \sum_{j=1}^{12} v_{\text{input},\text{int}(i),j} \tag{5.55}$$

where

- $pv_{\text{int}(i),\,n}$ = cumulative sum of volume proportions for leg associated with NEMA Phase n ($n = 2, 4, 6, 8$) at intersection i;
- $lv_{\text{int}(i),\,n}$ = leg volume (two-way total) for leg associated with NEMA Phase n at intersection i, vehicles per hour (veh/h);
- $tv_{\text{int}(i)}$ = total volume entering intersection i, veh/h; and
- $v_{\text{input},\,\text{int}(i),\,j}$ = movement j volume at intersection i (from input file), veh/h.

The leg location of the incident is determined by comparing a random number with the cumulative volume proportions. Using this technique, the likelihood of an incident being assigned to a leg is proportional to its volume, relative to the other leg volumes. The location is determined for a given intersection i by the following rule (Equation 5.56):

$$\begin{aligned}
&\text{Incident on Phase 2 if } Rv_{\text{int}(i),\text{con},\text{lan},\text{sev}} \le pv_{\text{int}(i),2} \\
&\text{Incident on Phase 4 if } pv_{\text{int}(i),2} < Rv_{\text{int}(i),\text{con},\text{lan},\text{sev}} \le pv_{\text{int}(i),4} \\
&\text{Incident on Phase 6 if } pv_{\text{int}(i),4} < Rv_{\text{int}(i),\text{con},\text{lan},\text{sev}} \le pv_{\text{int}(i),6} \\
&\text{Incident on Phase 8 if } pv_{\text{int}(i),6} < Rv_{\text{int}(i),\text{con},\text{lan},\text{sev}} \le pv_{\text{int}(i),8}
\end{aligned} \tag{5.56}$$

where $Rv_{\text{int}(i),\,\text{con},\,\text{lan},\,\text{sev}}$ is a random number for leg volume for intersection i, event type con, lane location lan, and severity sev.

Segment Location

When a specific segment is associated with an incident, the location of the incident is based on consideration of the volume in each direction of travel, *dv.* This volume is computed using the movement volume at the boundary intersection that uses NEMA Phase 2 to serve exiting through vehicles. The volume in the Phase 2 direction is computed as the sum of the movements exiting the segment at the boundary intersection (i.e., it equals the approach lane volume). The volume in the Phase 6 direction is computed as the sum of the movements entering the segment at the boundary intersection (i.e., it equals the departure lane volume). The two directional volumes are referenced to NEMA Phases 2 and 6. The sum of the two volumes equals the Phase 2 leg volume described in the previous subsection.

A cumulative volume proportion by direction is used to determine incident location. The calculation of these proportions is described by the following equations. One set of proportions is determined for the base input file and for each work zone and special event input file.

$$\begin{aligned}
pv_{\text{seg}(i),2} &= dv_{\text{seg}(i),2} / (dv_{\text{seg}(i),2} + dv_{\text{seg}(i),6}) \\
pv_{\text{seg}(i),6} &= 1.0
\end{aligned} \tag{5.57}$$

where

- $pv_{\text{seg}(i),\,n}$ = volume proportion for the direction of travel served by NEMA Phase n ($n = 2, 6$) on segment i; and

$dv_{seg(i),\,n}$ = directional volume for the direction of travel served by NEMA Phase n on segment i, veh/h.

The segment location of the incident is determined by comparing a random number with the cumulative volume proportions. Using this technique, the likelihood of an incident being assigned to a direction of travel is proportional to its volume, relative to the volume in the other direction. The location is determined for a given segment i by the following rule (Equation 5.58).

$$\text{Incident in Phase 2 direction if } Rv_{seg(i),con,lan,sev} \leq pv_{seg(i),2} \text{ Incident in Phase 6 direction if } pv_{seg(i),2} < Rv_{seg(i),con,lan,sev} \leq pv_{seg(i),6} \quad (5.58)$$

where $Rv_{seg(i),\,con,\,lan,\,sev}$ is equal to a random number for volume for segment i, event type con, lane location lan, and severity sev.

Step 6: Identify Analysis Period Incidents

Steps 3 through 5 are repeated for each hour of the subject day. As implied by the discussion to this point, all incidents are assumed to occur at the start of a given hour.

During this step, the analysis periods associated with an incident are identified. Specifically, each hour of the study period is examined to determine whether it coincides with an incident. If an incident occurs, then its event type, lane location, severity, and street location are identified and recorded. Each subsequent analysis period coincident with the incident is also recorded.

The incident duration from Equation 5.58 is rounded to the nearest hour for 1-hour analysis periods, or to the nearest quarter hour for 15-min analysis periods. This rounding is performed to ensure the most representative match between event duration and analysis period start/end times. This approach causes events that are shorter than one-half of the analysis period duration to be ignored (i.e., they will not be recognized in the scenario generation process). The use of a 15-min analysis period duration minimizes the number of events that are ignored, relative to a 1-hour analysis period.

Scenario File Generation Procedure

The scenario file generation procedure uses the results from the preceding three procedures to develop one urban streets engine input file for each analysis period in the reliability reporting period. As discussed previously, each analysis period is considered to be one scenario.

The sequence of calculations in the scenario file generation procedure is shown in Figure 5.19. The calculations and file generation proceed on a day-by-day and analysis-period-by-analysis-period basis in chronologic order. If a day is coincident with a work zone or special event, then the appropriate input file is loaded. Otherwise, the base input file is loaded.

Once loaded, the input file is modified to create a new input file for the subject analysis period. Modifications are made to the traffic volumes at each intersection and driveway. They are also made to the saturation flow rate at intersections influenced by an incident or a weather event. The speed is also adjusted for segments influenced by an incident or a weather event. Finally, the new input file is saved for evaluation in a subsequent stage of the reliability methodology.

The incident history developed by the traffic incident procedure is consulted during this procedure to determine if an incident occurs at an intersection or on a segment. If an incident occurs at an intersection, then the incident lane location data are consulted to determine which approach and movements are affected. If the incident occurs on the shoulder, then the shoulder in question is assumed to be the outside shoulder (as opposed to the inside shoulder). If a one-lane incident occurs, then the incident is assumed to occur in the outside lane. If a two-or-more-lane incident occurs, then the incident is assumed to occur in the outside two lanes.

It is also assumed that the incident occurs on the intersection approach lanes, as opposed to the departure lanes. This assumption is consistent with typical intersection crash patterns.

The scenario file generation procedure consists of a set of calculation steps. The calculations associated with each step are described in this section.

Step 1: Read Appropriate Input File

During this step, the appropriate input file is identified and read by the software. This step proceeds day-by-day and analysis-period-by-analysis-period in chronologic order. The date is used to determine whether a work zone or special event is present. If one is present, then the appropriate alternative input file is read. Otherwise, the base input file is read. The hour-of-day, day-of-week, and month-of-year demand adjustment factors associated with each file are also read (as identified previously in the traffic demand variation procedure).

Step 2: Compute Weather Adjustment Factors

Signalized Intersections

The HCM2010 Freeway Facilities chapter (Figure 10-15) provides capacity adjustment factors that are sensitive to rainfall intensity. Similar factors are provided that are sensitive to precipitation intensity during snowfall. A comparison of these factors with those reported by Agbolosu-Amison et al. (2004) and Perrin et al. (2001) for urban intersections indicates that these factors can also be used to adjust the intersection saturation flow rate.

Equation 5.59 replicates the trend in the HCM2010 freeway facility factors. It is used in Step 5 to estimate intersection saturation flow rate during weather events.

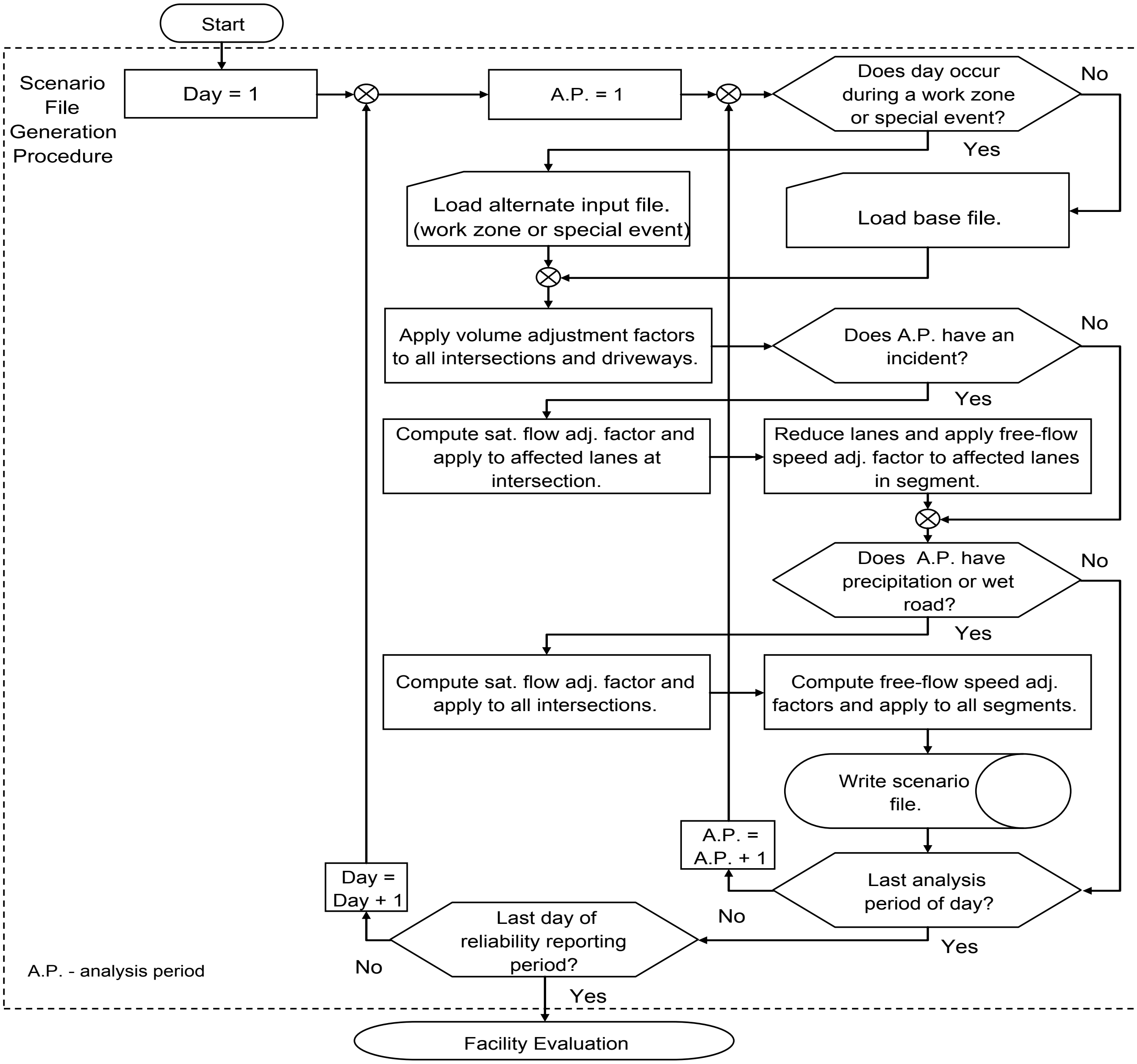


Figure 5.19. Scenario file generation procedure.

$$f_{rs,ap,d} = \frac{1.0}{1.0 + 0.48\,R_{r,ap,d} + 0.39\,R_{s,ap,d}} \qquad (5.59)$$

where

$f_{rs,ap,d}$ = saturation flow adjustment factor for rainfall or snowfall rs, during analysis period ap and day d;

$R_{r,ap,d}$ = rainfall rate during analysis period ap and day d, in./h; and

$R_{s,ap,d}$ = precipitation rate when snow is falling during analysis period ap and day d, in./h.

If Equation 5.59 is used for analysis periods with falling rain, then the variable R_s should equal 0.0. If it is used for analysis periods with falling snow, then the variable R_r should equal 0.0 and the variable R_s equals the precipitation rate (i.e., it is not a snowfall rate).

The factors obtained from Equation 5.59 apply when some precipitation is falling. If the pavement is wet and no rain is falling, then the adjustment factor is 0.95. If the pavement has snow or ice on it and snow is not falling, then the adjustment factor is 0.90. Each of these values is an average of the values reported by Agbolosu-Amison et al. (2004) and Perrin et al. (2001) for the corresponding conditions.

Equation 5.59 is not sensitive to the effect of driver familiarity with driving in snow. Some evidence suggests that drivers in cooler climates are less affected by snow than drivers in warmer climates. Additional research is needed to quantify this effect.

SEGMENTS

Perrin et al. (2001) also examined the effect of adverse weather on the average free-flow speed of an urban street. They did not measure rainfall or snowfall rates but they did stratify their data on the basis of weather conditions. The conditions cited included dry, wet, wet and snowing, wet and slushy, slushy in wheel path, snowy and sticking, and snowing and packed. Speed was observed to decline with each weather condition, in the order cited (i.e., speed was reduced for *wet,* it was further reduced for *wet and snowing,* and so forth). Research indicates that precipitation rate has an influence on speed that is functionally similar to that shown in Equation 5.59 (Rakha et al. 2008).

Equation 5.60 yields values that are consistent with those reported by Perrin et al. (2001) (based on some assumed snowfall rates for each weather condition) and Rakha et al. (2008). It is used in Step 7 to estimate the additional running time during weather events.

$$f_{s,rs,ap,d} = \frac{1.0}{1.0 + 0.48\,\mathrm{R}_{r,ap,d} + 1.4\,\mathrm{R}_{s,ap,d}} \tag{5.60}$$

where $f_{s,\,rs,\,ap,\,d}$ is equal to the free-flow speed adjustment factor for rainfall or snowfall during analysis period ap and day *d*.

If Equation 5.60 is used for analysis periods with falling rain, then the variable R_s should equal 0.0. If it is used for analysis periods with falling snow, then the variable R_r should equal 0.0 and the variable R_s equals the precipitation rate (i.e., it is not a snowfall rate).

The factors obtained from Equation 5.60 apply when some precipitation is falling. If the pavement is wet and no rain is falling, then the adjustment factor is 0.95. If the pavement has snow or ice on it and snow is not falling, then the adjustment factor is 0.90. Each of these values is based on the values reported by Perrin et al. (2001) for the corresponding conditions.

Step 3: Acquire Demand Adjustment Factors

During this step, the hour-of-day, day-of-week, and month-of-year demand adjustment factors associated with each analysis period are read (as identified previously in the traffic demand variation procedure). They are used in Step 6 to estimate the analysis period volumes.

Step 4: Compute Incident Adjustment Factors for Intersections

Incidents near the intersection have an influence on the number of lanes closed and on the saturation flow rate of the open lanes. The HCM2010 Freeway Facilities chapter (Figure 10-17) provides capacity adjustment factors that are sensitive to the number of basic lanes and the lane location of the incident. This effect is likely to be similar to that for urban street saturation flow rate.

Raub and Pfefer (1998) examined the effect of incident severity on the saturation flow rate on four-lane urban streets. Equation 5.61 replicates the trend in the HCM2010 freeway facility factors, but it is calibrated to the data reported by Raub and Pfefer.

$$f_{ic,int(i),n,m,ap,d} = \left(1.0 - \frac{N_{ic,int(i),n,m,ap,d}}{N_{n,int(i),n,m}}\right)\left(1.0 - \frac{b_{ic,int(i),n,ap,d}}{\sum_{m \in L,T,R} N_{n,int(i),n,m}}\right) \geq 0.10 \tag{5.61}$$

with

$$b_{ic,int(i),n,ap,d} = 0.58\, I_{fi,int(i),n,ap,d} + 0.42\, I_{pdo,int(i),n,ap,d} + 0.17\, I_{other,int(i),n,ap,d} \tag{5.62}$$

where

$f_{ic,\,int(i),\,n,\,m,\,ap,\,d}$ = saturation flow adjustment factor for incident presence for movement *m* (*m* = *L:* left, *T:* through, *R:* right) on leg associated with NEMA Phase *n* (*n* = 2, 4, 6, 8) at intersection *i* during analysis period ap and day *d*;

$N_{n,\,int(i),\,n,\,m}$ = number of lanes serving movement *m* on leg associated with NEMA Phase *n* at intersection *i*, lanes;

$N_{ic,\,int(i),\,n,\,m,\,ap,\,d}$ = number of lanes serving movement *m* blocked by the incident on leg associated with NEMA Phase *n* at intersection *i* during analysis period ap and day *d*, lanes;

$b_{ic,\,int(i),\,n,\,ap,\,d}$ = calibration coefficient based on incident severity on leg associated with NEMA Phase *n* at intersection *i* during analysis period ap and day *d*;

$I_{pdo,\,int(i),\,n,\,ap,\,d}$ = indicator variable for property-damage-only (PDO) crash on leg associated with NEMA Phase *n* at intersection *i* during analysis period ap and day *d* (= 1.0 if PDO crash, 0.0 otherwise);

$I_{fi,\,int(i),\,n,\,ap,\,d}$ = indicator variable for fatal-or-injury crash on leg associated with NEMA Phase *n* at intersection *i* during analysis period ap and day *d* (= 1.0 if fatal-or-injury crash, 0.0 otherwise); and

$I_{other,\,int(i),\,n,\,ap,\,d}$ = indicator variable for noncrash incident on leg associated with NEMA Phase *n* at intersection *i* during analysis period ap and day *d* (= 1.0 if noncrash incident, 0.0 otherwise).

Equation 5.61 is applied to each approach traffic movement. For a given movement, the first term of Equation 5.61

adjusts the saturation flow rate on the basis of the number of lanes blocked by the incident. If the incident is located on the shoulder or in the lanes associated with another movement m (i.e., $N_{ic} = 0$), then this term equals 1.0.

The second term of Equation 5.61 represents the adjustment for incident presence on the approach, and Equation 5.62 incorporates the adjustment into this term to account for incident severity. The variable b_{ic} represents the equivalent number of lanes lost as a result of the incident.

Equation 5.61 does not include sensitivity to the distance between the incident location and the downstream intersection. The incident's effect on saturation flow rate will likely be reduced if the incident is located further back on the approach. Additional research is needed to quantify this effect.

Equation 5.61 is used for each movement to estimate the saturation flow rate adjustment factor for incidents. If all lanes associated with a movement are closed because of the incident, then an adjustment factor of 0.10 is used. This approach effectively closes the lane but does not remove it from the intersection analysis. Changes to the approach lane allocation in an urban streets engine input file can be problematic because the engine recognizes only specific combinations of lane assignment, phasing sequence, left-turn mode, and volume. A change to the number of approach lanes could lead to an unrecognized combination and calculation failure.

Step 5: Compute Saturation Flow Rate for Intersections

During this step, the saturation flow rate for each intersection movement is adjusted using the factors computed in Steps 2 and 4. The weather adjustment factor is applied to all movements at all intersections. The incident adjustment factor is applied only to the movements affected by an incident.

The weather and incident factors are multiplied by the saturation flow rate in the input file to produce a revised estimate of the saturation flow rate.

Step 6: Compute Traffic Demand Volumes

During this step, the volume for each movement is adjusted using the appropriate hour-of-day, day-of-week, and month-of-year factors to estimate the average hourly flow rate for the subject analysis period. Equation 5.63 is used for this purpose.

$$v_{\text{int}(i),j,h,d} = \frac{v_{\text{input,int}(i),j}}{f_{\text{hod,input}}\ f_{\text{dow,input}}\ f_{\text{moy,input}}} f_{\text{hod},h,d}\ f_{\text{dow},h,d}\ f_{\text{moy},h,d} \qquad (5.63)$$

where

$v_{\text{int}(i),j,h,d}$ = adjusted hourly flow rate for movement j at intersection i during hour h and day d, veh/h;

$v_{\text{input,int}(i),j}$ = movement j volume at intersection i (from input file), veh/h;

$f_{\text{hod},h,d}$ = hour-of-day adjustment factor based on hour h and day d;

$f_{\text{dow},h,d}$ = day-of-week adjustment factor based on day d;

$f_{\text{moy},h,d}$ = month-of-year adjustment factor based on day d;

$f_{\text{hod,input}}$ = hour-of-day adjustment factor for hour and day associated with v_{input};

$f_{\text{dow,input}}$ = day-of-week adjustment factor for day associated with v_{input}; and

$f_{\text{moy,input}}$ = month-of-year adjustment factor for day associated with v_{input}.

If a 15-min analysis period is used, then the adjusted hourly flow rate is applied to all four analysis periods coincident with the subject hour h. Equation 5.63 is also used to adjust the volumes associated with each driveway on each segment.

Random Variation Among 15-Min Periods

If a 15-min analysis period is used, the analyst has the option of adding a random element to the adjusted hourly volume for each movement and analysis period. Including this random variation provides a more realistic estimate of performance measure variability. However, it ensures that every analysis period is unique (thus making it less likely that similar scenarios can be found for the purpose of reducing the total number of scenarios to be evaluated). If this option is applied, then the turn movement volumes at each signalized intersection are adjusted using a random variability based on the peak-hour factor. Similarly, the turn movement volumes at each driveway are adjusted using a random variability based on a Poisson distribution.

The relationship between the peak-hour factor and analysis period flow rate is shown in Figure 5.20. The data used to

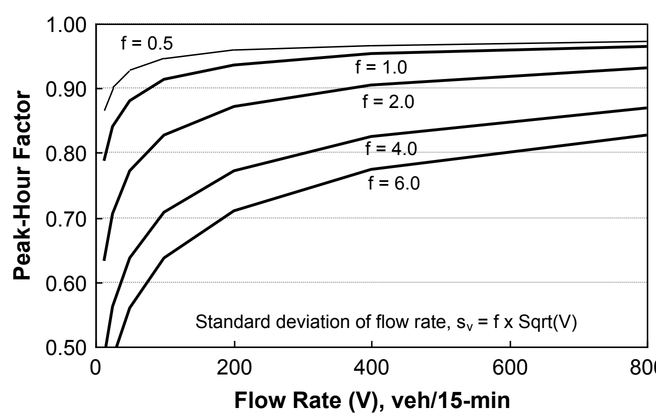


Figure 5.20. Relationship between peak-hour factor and flow rate. Sqrt = square root.

develop the trend lines shown in this figure were based on simulated flow rates for 144 hours at each of seven average flow rates. The flow rate for each 15-min period in a given hour was computed using Monte Carlo methods with a gamma distribution, and a standard deviation that computed as factor f times the square root of the flow rate. For each simulated hour and flow rate combination, one peak-hour factor was computed. The average of these 144 observations was then added to the database along with the associated with the flow rate. The process was repeated for values of factor f ranging from 0.2 to 7.

The following relationship (Equation 5.64) was fit to the data underlying Figure 5.20. The R^2 for the model is 0.996. The peak-hour factor is provided by the analyst.

$$f_{\text{int}(i),j,h,d} = \frac{1.0 - \text{PHF}_{\text{int}(i)}}{\text{PHF}_{\text{int}(i)}} \sqrt{0.25\, v_{\text{int}(i),j,hd}} \exp\left(-0.00679 + 0.004\ \text{PHF}^{-4}_{\text{int}(i)}\right) \tag{5.64}$$

where

$f_{\text{int}(i),j,h,d}$ = adjustment factor used to estimate the standard deviation of demand flow rate for movement j at intersection i during hour h and day d; and

$\text{PHF}_{\text{int}(i)}$ = peak-hour factor for intersection i.

Equation 5.65 is used to compute the randomized hourly flow rates for each movement at each signalized intersection.

$$v^*_{\text{int}(i),j,\text{ap},d} = 4.0 \times \text{gamma}^{-1}\begin{pmatrix} p = \text{Rf}_{\text{ap},d},\ \mu = 0.25\, v_{\text{int}(i),j,h,d}, \\ \sigma = f_{\text{int}(i),j,h,d}\sqrt{0.25\, v_{\text{int}(i),j,h,d}} \end{pmatrix} \tag{5.65}$$

where

$v^*_{\text{int}(i),j,\text{ap},d}$ = randomized hourly flow rate for movement j at intersection i during analysis period ap and day d, veh/h; and

$\text{Rf}_{\text{ap},d}$ = random number for flow rate for analysis period ap and day d.

Similarly, Equations 5.66 and 5.67 are used to compute the randomized hourly flow rates for each driveway. The first equation is used if the adjusted hourly flow rate is 64 veh/h or less. The second equation is used if the flow rate exceeds 64 veh/h.

If $v_{\text{int}(i),j,h,d} \le 64$ veh/h, then

$$v^*_{\text{int}(i),j,\text{ap},d} = 4.0 \times \text{Poisson}^{-1}\left(p = \text{Rf}_{\text{ap},d},\ \mu = 0.25\, v_{\text{int}(i),j,h,d}\right) \tag{5.66}$$

Otherwise,

$$v^*_{\text{int}(i),j,\text{ap},d} = 4.0 \times \text{normal}^{-1}\begin{pmatrix} p = \text{Rf}_{\text{ap},d},\ \mu = 0.25\, v_{\text{int}(i),j,h,d}, \\ \sigma = \sqrt{0.25\, v_{\text{int}(i),j,h,d}} \end{pmatrix} \tag{5.67}$$

where $\text{Poisson}^{-1}(p, \mu)$ is the value associated with probability p for the cumulative Poisson distribution with mean μ.

Step 7: Compute Speed for Segments

Additional Delay

During this step, the effect of incidents and weather on segment speed is determined. The structure of the urban streets engine (and its input file) is such that the adjustment is most easily introduced as an additional delay incurred along the segment. The variable d_{other} in Equation 17-6 of the HCM2010 is used with this approach. This variable is available for modification in the input file. The new value is computed using Equations 5.68, 5.69, and 5.70.

$$d_{\text{other},\text{seg}(i),n,\text{ap},d} = L_{\text{seg}(i)}\left(\frac{1.0}{S^*_{\text{fo},\text{seg}(i),n,\text{ap},d}} - \frac{1.0}{S_{\text{fo},\text{seg}(i),n}}\right) \tag{5.68}$$

with

$$S^*_{\text{fo},\text{seg}(i),n,\text{ap},d} = S_{\text{fo},\text{seg}(i),n} \times f_{s,\text{rs},\text{ap},d} \times \left(1.0 - \frac{b_{\text{ic},\text{seg}(i),n,\text{ap},d}}{N_{o,\text{seg}(i),n}}\right) \tag{5.69}$$

$$b_{\text{ic},\text{seg}(i),n,\text{ap},d} = 0.58\, I_{\text{fi},\text{seg}(i),n,\text{ap},d} + 0.42\, I_{\text{pdo},\text{seg}(i),n,\text{ap},d} + 0.17\, I_{\text{other},\text{seg}(i),n,\text{ap},d} \tag{5.70}$$

where

$d_{\text{other},\text{seg}(i),n,\text{ap},d}$ = additional delay for the direction of travel served by NEMA Phase n (n = 2, 6) on segment i during analysis period ap and day d, seconds per vehicle (s/veh);

$L_{\text{seg}(i)}$ = length of segment i, ft;

$S_{\text{fo},\text{seg}(i),n}$ = base free-flow speed for the direction of travel served by NEMA Phase n on segment i, ft/s;

$S^*_{\text{fo},\text{seg}(i),n,\text{ap},d}$ = adjusted base free-flow speed for the direction of travel served by NEMA Phase n on segment i during analysis period ap and day d, ft/s;

$b_{\text{ic},\text{seg}(i),n,\text{ap},d}$ = calibration coefficient based on incident severity on leg associated with NEMA Phase n at intersection i during analysis period ap and day d;

$N_{o,\text{seg}(i),n}$ = number of lanes serving direction of travel served by NEMA Phase n on segment i, lanes;

$I_{\text{pdo},\text{seg}(i),n,\text{ap},d}$ = indicator variable for property-damage-only (PDO) crash in the direction of travel served by NEMA Phase n on segment i during analysis period ap and day d (= 1.0 if PDO crash, 0.0 otherwise);

$I_{\text{fi, seg}(i),\ n,\ \text{ap},\ d}$ = indicator variable for fatal-or-injury crash in the direction of travel served by NEMA Phase n on segment i during analysis period ap and day d (= 1.0 if fatal-or-injury crash, 0.0 otherwise); and

$I_{\text{other, seg}(i),\ n,\ \text{ap},\ d}$ = indicator variable for noncrash incident in the direction of travel served by NEMA Phase n on segment i during analysis period ap and day d (= 1.0 if noncrash incident, 0.0 otherwise).

The term in parentheses in Equation 5.69 is a speed adjustment factor that reflects the average speed adjacent to the incident. It is a conservative estimate of incident effect on speed when the segment is long, relative to the length of segment on which traffic speed is actually influenced by the incident. Additional research is needed to determine if the *effective incident length* is helpful in estimating segment speed and to incorporate this effect in Equation 5.69.

The calibration coefficients in Equation 5.70 are the same as those used in Equation 5.62 to estimate the saturation flow rate adjustment factor. The speed-flow model used for urban streets in the HCM2010 (i.e., Equation 17-5) is based on the assumption that segment capacity is directly proportional to the free-flow speed. This relationship indicates that any situation that reduces capacity (or saturation flow rate) by a fixed percentage will also reduce speed by the same percentage. Additional research is needed to confirm the rationale for this adjustment.

The delay estimated from Equation 5.68 is added to the *other delay* variable in the input file to produce a combined *other delay* value for segment running speed estimation.

Segment Lane Closure

If an incident is determined to be located in one or more lanes, then the variable for the number of through lanes on the segment is reduced accordingly. This adjustment is made for the specific segment and direction of travel associated with the incident.

The variable indicating the number of major-street through lanes at each driveway is reduced in a similar manner when the incident occurs on a segment and closes one or more lanes. This adjustment is made for each driveway on the specific segment affected by the incident.

Table 5.29. Additional Critical Left-Turn Headway Depending on Weather

Weather Condition	Additional Critical Left-Turn Headway (s)	
	Based on Zohdy et al. (2011)	Recommended
Clear, snow on pavement	0.92	0.9
Clear, ice on pavement	0.76	0.9
Clear, water on pavement	1.15	0.7
Snowing	1.25	1.2
Raining	0.68	0.7

Step 8: Adjust Critical Left-Turn Headway

Research indicates that the critical headway for left-turn drivers increases by 0.7 to 1.2 seconds, depending on the type of weather event and the opposing lane associated with the conflicting vehicle. The difference between the critical headway values for various weather conditions and that for fair weather was computed from the recommended values (Zohdy et al. 2011). These differences were computed for each combination of weather condition and critical path. The computed average for each weather condition is listed in the second column of Table 5.29.

The trends in column two of Table 5.29 are logical, with the exception that the value for *clear, water on pavement* is larger than that for *raining*. Intuitively, the reverse trend would be more realistic, as found when comparing *snowing* with *clear, snow on pavement*. The recommended values, based on the examination of the Zohdy et al. (2011) data, are listed in the last column of Table 5.29. These values follow the trends shown in column two, except the value for *clear, water on pavement*, which was set equal to that for *raining* (i.e., 0.7) because a larger value is counterintuitive.

Step 9: Save Scenario File

During this step, the input file with the updated values is saved for evaluation in the next stage of the reliability methodology. The file name used for the new file is the same as that for the original input file, but it is preceded by the date and time associated with the subject analysis period.

CHAPTER 6

Model Enhancements

This chapter discusses the enhancements that have been made to the FREEVAL and STREETVAL models during this project. It is divided into the following five sections:

- Freeway facilities introduction;
- Description of freeway facility enhancements;
- FREEVAL-RL calibration;
- Summary of freeway model enhancements; and
- Urban streets enhancements.

Freeway Facilities Introduction

The FREEVAL (FREeway EVALuation) tool was first developed as a computational engine in 2000 for the HCM chapter on freeway facilities methodology. It has since gone through several improvements, and the most recent, FREEVAL 2010, is implemented in a Microsoft Excel and Visual Basic programming platform. FREEVAL 2010 is fully compatible with the HCM2010 and is distributed to HCM users via the Volume 4 website (TRB 2010b). It is designed for the analysis of freeway facilities but incorporates all methodological details for the analysis of basic freeway segments, merge and diverge sections, and freeway weaving segments. It is unique in the HCM2010 in that it can analyze both undersaturated and congested regimes and allows for the evaluation of multiple segments across multiple analysis time periods. Since the publication of HCM2010, FREEVAL 2010 has been customized to create a more user-friendly environment for analyzing work zones (NCDOT FREEVAL-WZ), as well as for managed lanes (FREEVAL-ML) through NCHRP Project 3-96 (Wang et al. 2012).

The adaptation of the freeway facilities method for use in a reliability analysis required several changes and enhancements to FREEVAL. The objective of this section is to provide an overview of the recent reliability analysis enhancements to both the core computational engine and the user interface. The enhanced computational engine is named FREEVAL-RL (FREeway EVALuation–ReLiability).

Incorporation of the Two-Capacity Phenomenon under Queue Discharge Conditions

The HCM2010's freeway facilities methodology does not consider the two-capacity phenomenon, namely the drop in throughput from theoretical capacity that occurs after breakdown at a freeway bottleneck. The enhanced FREEVAL-RL accounts for this capacity reduction in the queue discharge based on a user-defined proportional drop in capacity, designated as α. The default for α is set at 7% on the basis of a recent synthesis of the literature.

Incorporation of Speed Adjustment Factor for Some Nonrecurring Congestion Sources

In the HCM2010, operational impacts of nonrecurring congestion sources were addressed through a capacity adjustment factor (CAF), which reduces the basic segment capacity by a multiplicative factor. Using Equation 25-1 in the HCM2010, the methodology develops a new speed-flow curve between the free-flow speed and the new, user-defined capacity. In Project L08, the speed adjustment factor (SAF) amends Equation 25-1 by also reducing the free-flow speed (intercept of the speed-flow curve). This enhancement is critical for modeling weather impacts, which have been demonstrated to result in significant reductions in speed, even under low-volume conditions.

Improved Modeling of CAF and SAF for Merge, Diverge, and Weaving Segments

In the HCM2010, the application of Equation 25-1 does not distinguish between segment types. Specifically, when using a CAF or SAF, the analyst essentially assumed that the adjusted segment is a basic segment. In FREEVAL's computations, the method bypasses weaving and ramp-segment speed calculation procedures and uses Equation 25-1 instead. This approach sometimes results in unrealistic speed estimates and inconsistent results, as segment speeds may actually increase when adding a CAF. In Project L08, the enhancements directly incorporate SAF and CAF into the respective procedure for each segment type and thus consistently account for the particular segment characteristics.

New Defaults for CAF and SAF for Incident and Weather Events on Freeways

With the introduction of SAF and the increased use of CAF and SAF in evaluating nonrecurring sources of congestion in a reliability context, national defaults values should be offered to encourage a uniform and consistent application of the methodology across agencies. Through an extensive literature review, the research team developed new default values for CAF and SAF, which have been incorporated in the methodology. Note that CAF inputs for work zones are adapted directly from HCM2010, pending the results of ongoing research in NCHRP Project 3-107, Work Zone Capacity Methods for the HCM.

Enhanced Performance Measures for Congested Conditions

To make the computational engine reliability analysis ready, two reliability performance measures were added to the engine's output. The first of these performance measures is the travel time index (TTI), which is used for deriving the travel time distribution. The augmented analysis also reports the denied entry vehicle queue length, which describes vehicles stored in a queue upstream of the first analysis segment.

Computational Automation

To generate a travel time distribution, FREEVAL needs to be executed multiple times, with a distinct FREEVAL run performed for each scenario, reflecting each scenario's unique combination of FREEVAL input data. Running FREEVAL in a manual mode to generate travel time distributions is very time-consuming. Therefore, automating the scenario runs is a necessary addition to the computational engine. The revised FREEVAL-RL engine does so by automatically interacting with the freeway scenario generator (FSG) and directly receiving scenario-specific input for performance measure computation. FREEVAL-RL also provides automated generation of standardized reliability outputs.

Travel Time Index Distribution Calibration

In the next step, FREEVAL was calibrated for generating TTI distributions for HCM freeway reliability analysis. For this purpose, three calibration parameters were identified: the overall demand-level adjustment from the seed file, the percent drop in capacity during breakdown, and the jam density. The calibration parameter effects on the TTI distribution were studied for a 12.5-mile facility (I-40 EB in Raleigh, North Carolina) for which segment and facility travel times for the calendar year 2010 were available from INRIX, the traffic data service. On the basis of the initial model runs and previous studies, three candidate values for each calibration parameter were selected, resulting in 27 distinct combinations. To minimize calibration bias, the calibration analysis was limited to conditions in which no incidents or inclement weather events were evident.

The results of a two-sample Kolmogorov–Smirnov (KS) test indicated that increasing the originally estimated demand levels in the seed file by 3% and selecting a value of 9% for the queue discharge capacity drop yielded estimated TTI distributions that were not significantly different from the empirical INRIX distribution. Jam density values showed very little effect on the resulting TTI distribution.

Description of Freeway Facility Enhancements

In this section, each of the enhancements is explained in more detail with the exception of travel time index distribution calibration.

The section starts with the consideration of the two-capacity phenomenon and continues with SAF and CAF adjustments for basic segments, as well as merge, diverge, and weaving segments. New default values for CAF and SAF are presented followed by a discussion of recently added performance measures. The section ends with a high-level explanation of the automation process.

Incorporation of the Two-Capacity Phenomenon

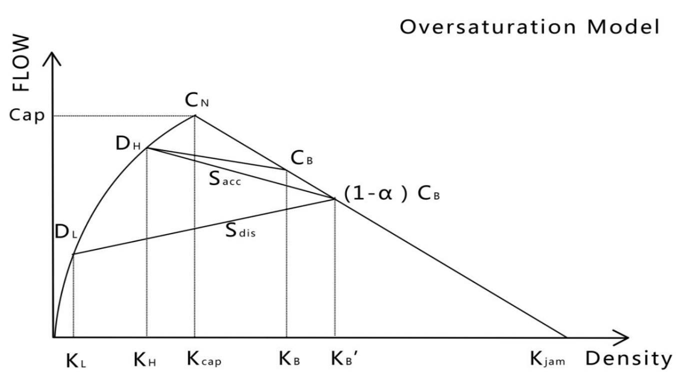
The HCM2010 freeway facilities methodology encompassed undersaturated and congested flow regimes over multiple time periods. However, the methodology was limited by its assumption of a fixed capacity threshold between the two flow regimes. The method did not consider the drop in throughput from theoretical capacity that has been observed after breakdown has occurred at freeway bottlenecks. In other words, in the HCM2010 methodology, when demand exceeds capacity at a freeway bottleneck, queuing and congestion impacts are estimated, but the bottleneck discharges traffic at the prebreakdown capacity. However, strong evidence in the literature suggests that the freeway capacity at bottlenecks is measurably reduced after breakdown has occurred. Many studies have focused on the topic of queue discharge flow, and their results confirm that the capacity at a bottleneck drops by a factor ranging from 1% to 18%, with an average reduction around 7%. This finding is often referred to as the two-capacity phenomenon.

Past research has demonstrated that the incorporation of the freeway two-capacity phenomenon will result in nontrivial impacts on performance measures such as queue lengths, queue formation and dissipation times, speed and travel time, and facility levels of service.

At first glance, a 5% to 7% reduction in capacity may seem trivial. Such a capacity reduction is equivalent to a drop of 120 vehicles per hour (veh/h) in capacity for a high-design freeway lane (2,400 passenger cars per hour per lane, or pcphpl). However, a closer investigation of shock wave theory for congested flow on freeways reveals that the drop in capacity of this magnitude will have significant impact on oversaturated traffic conditions.

Figure 6.1 shows a shock wave diagram of traffic flow which is conceptually similar to the freeway facilities method adopted in the HCM2010. At density values below the density at capacity (K_{cap}), or demand values below the segment capacity (C_N), the model uses the speed–flow relationship in the HCM2010 segment chapters for basic freeway segments, merge segments, diverge segments, or weaving segments. For densities above capacity (45 passenger cars per mile per lane, or pcpmpl), a linear flow–density model is assumed. The model is used to estimate the shock wave speed upstream of an active bottleneck (S_b) with capacity less than the high upstream demand ($C_B < D_H$). The diagram also shows the speed of the accumulating shock wave (S_{acc}), and the dissipating shock wave (S_{dis}) after the upstream demand has dropped below the bottleneck capacity ($C_B > D_L$).

When the bottleneck is activated, the maximum flow that is allowed to travel through the segment equals the downstream bottleneck capacity C_B in the current procedure. With consideration of the two-capacity regime, the actual throughput in the bottleneck after breakdown is assumed to drop by α%, where (1 − α) is the fraction of remaining bottleneck



Source: Hu et al. (2012), Figure 1, p. 79. Reproduced with permission of the Transportation Research Board.

Figure 6.1. Shock wave illustration with two-capacity approach.

capacity. This is indicated in Figure 6.1 as a reduction in flow and an increase in density from K_B to K'_B. In Figure 6.1, the slope S_{acc} represents the speed of the forming queue (shock wave speed) under a demand flow D_H. It can be computed by using Equation 6.1:

$$S_{acc} = \frac{D_H - C_B(1-\alpha)}{K_H - K'_B} \tag{6.1}$$

where

D_H = demand flow rate upstream of the queue;
C_B = uninterrupted bottleneck capacity;
K_H = density upstream of the queue;
K'_B = density in the queue during queue discharge; and
α = percent capacity drop (fraction).

The shock wave speed is a critical variable in the oversaturation analysis. It helps predict the dimension of the queue, which greatly affects other freeway traffic characteristics and performance measures, such as density, speed, and travel time on the facility. To ascertain the impact caused by the capacity drop, expressing S_{acc} based on α is useful. Using the similar triangle rule, Equation 6.2 can be used to show the density during queue discharge.

$$K'_B = K_{jam} - (1-\alpha) \times \frac{C_B}{C_N} \times (K_{jam} - K_{cap}) \tag{6.2}$$

Substituting Equation 6.2 into Equation 6.1 gives the speed of the accumulating wave (Equation 6.3):

$$S_{acc} = \frac{C_N \times F_H - C_N \times C_B(1-\alpha)}{C_N \times (K_H - K_{jam}) + (1-\alpha) \times C_B \times (K_{jam} - K_{cap})} \tag{6.3}$$

When the peak period is over, demand is expected to decrease and eventually drop below the bottleneck capacity. At that time, the queue will start to dissipate. In Figure 6.1, the slope S_{dis} represents the speed of queue dissipation. Similarly, Equation 6.4 provides the speed of the dissipating wave, S_{dis}.

$$S_{dis} = \frac{C_N \times F_L - C_N \times C_B(1-\alpha)}{C_N \times (K_L - K_{jam}) + (1-\alpha) \times C_B \times (K_{jam} - K_{cap})} \tag{6.4}$$

Equations 6.3 and 6.4 make clear that the shock wave speeds (accumulating or dissipating) are sensitive to a number of parameters, including the magnitude of capacity drop, bottleneck capacity, normal segment capacity and demand flow rate, and other segment attributes such as jam density and density at capacity. The impact from capacity drop would thus vary with the characteristics of the freeway segment of interest in a nonlinear fashion.

In summary, the effect of capacity drop in queue discharge mode is not limited to decreasing the bottleneck capacity. It also increases queue formation shock wave speed and decreases the queue dissipation speed. In past research, even a 5% drop in capacity has been shown to result in an approximate 80% increase in queue length and 40% increase in travel time for a simulated test facility. In the proposed implementation for HCM2010, a default queue discharge drop of 7% is proposed on the basis of research. However, the user is given the flexibility in the FREEVAL-RL engine to set α within a range of 0% to 10%. The research team also expects α to emerge as a key calibration factor in the methodology, as described later in this section.

Incorporation of Speed Adjustment Factor for Basic Segments

The effects of weather and incidents on freeway facilities are modeled through a capacity adjustment factor (CAF) in the HCM2010. However, strong evidence in the literature suggests that weather and incidents also affect the free-flow speed, with especially severe weather events like heavy rain and snow resulting in significant speed drops even at very low volume levels. Therefore, another input was needed to account for free-flow speed adjustment as a result of the congestion source. This new adjustment factor is the speed adjustment factor (SAF).

The research team explored various options for incorporating SAF into the HCM2010 methodology and ultimately developed a modification to Equation 25-1. That equation dates back to the HCM2000 and uses a CAF to estimate a revised speed–flow relationship for work zones and incidents. Inputs to the equation are the base capacity (C), the free-flow speed (FFS), the CAF, and the prevailing flow rate (v_p). The updated Equation 6.5, which adds SAF as a multiplier of the FFS, follows:

$$S = (\text{FFS} \times \text{SAF}) + \left[1 - e^{\ln\left((\text{FFS}\times\text{SAF})+1-\frac{C\times\text{CAF}}{45}\right)\times\frac{v_p}{C\times\text{CAF}}}\right] \tag{6.5}$$

where

S = segment speed, mi/h;
FFS = segment free-flow speed, mi/h;
SAF = segment speed adjustment factor;
C = original segment capacity, pcphpl;
CAF = capacity adjustment factor; and
v_p = segment flow rate, pcphpl.

With the revised Equation 25-1, the HCM2010 results remain unchanged for cases with SAF = 1.0, which may include some work zone configurations. The introduction of SAF results in internally consistent results and provides an additional calibration tool to enable better fitting to local conditions and driver culture.

An example application of SAF and CAF for different base free-flow speeds and weather categories is shown in Figure 6.2. The defaults for SAF and CAF are based on a new research synthesis (presented in a later section). The graph shows the effects of medium rain (dashed) and heavy snow (dotted), relative to clear weather conditions (solid line) for base free-flow speeds of 75 mph (blue), 65 mph (green), and 55 mph (red).

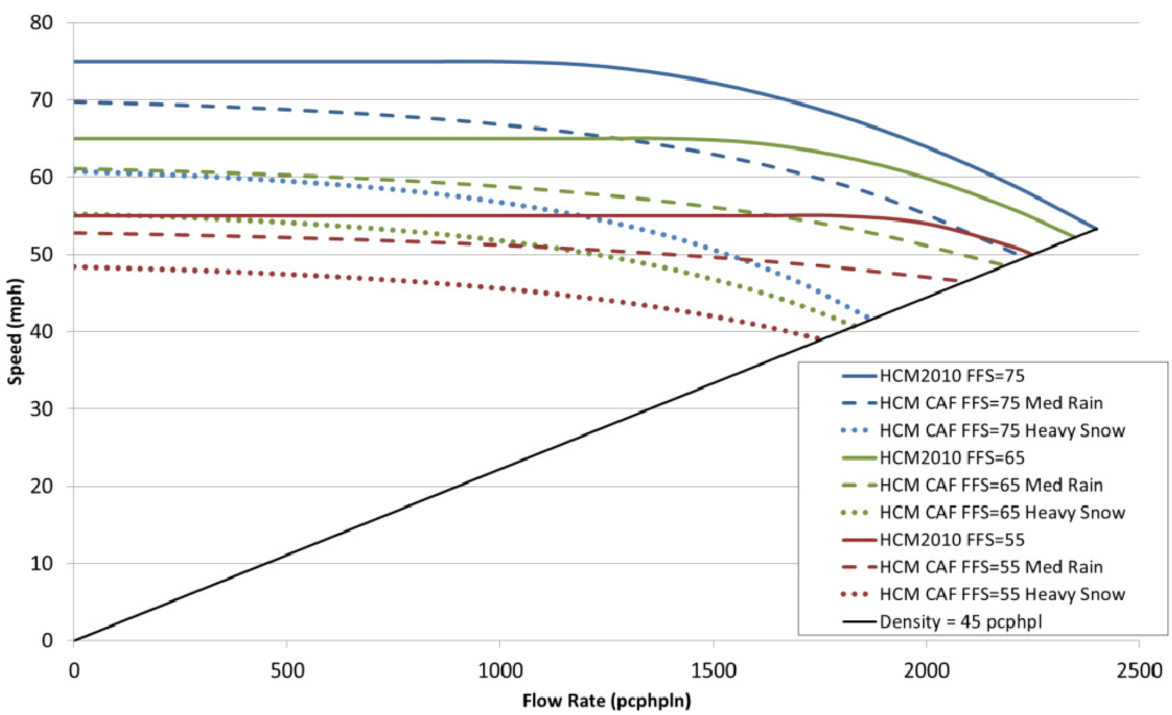


Figure 6.2. Example application of SAF and CAF for different base FFS and weather categories.

Table 6.1. Estimating Speed at Merge (On-Ramp) Junctions with SAF Consideration

Average Speed in	Equation	
Ramp influence area	$S_R = (FFS \times SAF) - ((FFS \times SAF) - 42)\ M_S$ $M_S = 0.321 + 0.0039e^{(v_{R12}/1{,}000)} - 0.002\ (L_A S_{FR} \times SAF/1{,}000)$	
Outer lanes of freeway	$S_O = FFS \times SAF$	$v_{OA} < 500$ pc/h
	$S_O = (FFS \times SAF) - 0.0036(v_{OA} - 500)$	500 pc/h $\leq v_{OA} \leq$ 2,300 pc/h
	$S_O = (FFS \times SAF) - 6.53 - 0.006(v_{OA} - 2{,}300)$	$v_{OA} >$ 2,300 pc/h

Consideration of CAF and SAF for Other Segment Types

The equation for CAF and SAF described in the previous section is ultimately intended for application to basic freeway segments. However, in the HCM2000 and HCM2010, it was also applied to the analysis of merge, diverge, and weaving segments with CAFs less than 1.0. As a further improvement, this section describes the adaptation of CAF and SAF to these other HCM2010 segment types.

A challenge arises in both ramp (merge or diverge) and weaving segment analysis when considering CAF and SAF because the methodologies for both of these freeway segment categories do not use segment capacity as an input to the speed prediction equation. In essence, the HCM2010 procedures for these segment types violate the fundamental equation of traffic flow (speed = flow × density). Both methods first estimate segment capacity and then perform a check to assure that traffic demands are below that capacity (otherwise, demand-to-capacity >1 and the oversaturated module is invoked). If the segment passes the capacity check, the segment speed is estimated from an independent regression equation. With the L08 enhancements, the base capacity is adjusted with the appropriate CAF before performing the demand-to-capacity check. Equation 6.6 shows how the adjusted capacity is calculated:

$$\text{Adjusted Capacity} = \text{Base Capacity} \times \text{CAF} \tag{6.6}$$

where

Adjusted Capacity = capacity used to perform the demand-to-capacity check to switch to the oversaturated procedure (if demand-to-capacity >1, then the oversaturated procedure is invoked);

Base Capacity = segment capacity estimated from the appropriate HCM2010 chapter; and

CAF = user input capacity adjustment factor.

Given the current structure of the HCM methodology, the research team implemented CAF and SAF separately. Specifically, CAF is used as a multiplicative factor of the segment base capacity in the initial checks, while SAF is subsequently used as a multiplier of FFS in the speed prediction equation (discussed for merge/diverge and weaving segments in the following subsection). Principally, the application of CAF and SAF is consistent with the basic segment procedure, with the caveat that the factors are applied in two (or more) separate steps.

Merge and Diverge Segments

Exhibit 13-11 in the HCM2010 gives equations for estimating the average speed of vehicles within the ramp influence area, as well as in outer lanes of the freeway. Those equations are updated by this research to incorporate the SAF. The updated equations are shown in Table 6.1.

Exhibit 13-12 in the HCM2010 is used to estimate speed at off-ramp (diverge) junctions in a way that is similar to how Exhibit 13-11 is used to estimate speed at on-ramp segments. The updated equations are shown in Table 6.2.

The variables in Table 6.1 and Table 6.2 are defined as follows:

S_R = average speed of vehicles within the ramp influence area, mph; for merge areas this includes all ramp and freeway vehicles in lanes 1 and 2; for diverge areas, this includes all vehicles in lanes 1 and 2;

Table 6.2. Estimating Speed at Diverge (Off-Ramp) Junctions with SAF Consideration

Average Speed in	Equation	
Ramp influence area	$S_R = (FFS \times SAF) - ((FFS \times SAF) - 42)D_S$ $D_S = 0.883 + 0.00009v_R - 0.013(S_{FR} \times SAF)$	
Outer lanes of freeway	$S_O = 1.097(FFS \times SAF)$	$v_{OA} <$ 1,000 pc/h
	$S_O = 1.097(FFS \times SAF) - 0.0039\ (v_{OA} - 1{,}000)$	$v_{OA} \geq$ 1,000 pc/h

Table 6.3. Default Values Used in Merge (On-Ramp) Segment Analysis

FFS (mph)	S_{FR} (mph)	L_A (ft)	Normal		Medium Rain		Heavy Snow	
			SAF	CAF	SAF	CAF	SAF	CAF
75	45	1,500	1	1	0.93	0.90	0.81	0.72
65	45	1,500	1	1	0.94	0.92	0.85	0.76
55	45	1,500	1	1	0.96	0.94	0.88	0.80

S_O = average speed of vehicles in outer lanes of the freeway, adjacent to the 1,500-ft ramp influence area, mph;
S = average speed of all vehicles in all lanes within the 1,500-ft length covered by the ramp influence area, mph;
FFS = free-flow speed of the freeway, mph;
SAF = segment speed adjustment factor of the ramp segment;
S_{FR} = free-flow speed of the ramp, mph;
L_A = length of acceleration lane, ft;
v_R = demand flow rate on ramp, pcph;
v_{12} = demand flow rate in lanes 1 and 2 of the freeway upstream of the ramp influence area;
v_{R12} = total demand flow rate entering the on-ramp influence area, including v_{12} and v_R, pcph;
v_{OA} = average per-lane demand flow in outer lanes adjacent to the ramp influence area (not including flow in lanes 1 and 2), pcphpl;
M_s = speed index for on-ramps (merge areas); this is simply an intermediate computation that simplifies the equations; and
D_s = speed index for off-ramps (diverge areas); this is simply an intermediate computation that simplifies the equations.

By using Exhibit 13-13 in the HCM2010, the average speeds for merge and diverge (on-ramp and off-ramp) junctions are calculated. Similar to basic segments, a sensitivity analysis was performed for a typical merge (on-ramp) segment. The default values used in this analysis are shown in Table 6.3.

Figure 6.3 shows the impacts of various weather events, including medium rain (dashed) and heavy snow (dotted), relative to normal weather conditions (solid line) for base free-flow speeds of 75 mph (blue), 65 mph (green), and 55 mph (red) on a typical merge (on-ramp) segment. Each line in Figure 6.3 terminates at the capacity for the prevailing adjusted FFS and weather conditions. The flow rate on the x-axis is referenced to the segment immediately downstream of the on-ramp.

Weaving Segments

The capacity of a weaving segment is calculated using Equation 12-3 in HCM2010. In the L08 enhancements, the weaving segment capacity is further adjusted by the appropriate CAF if necessary (Equation 6.6). Similar to ramp segments,

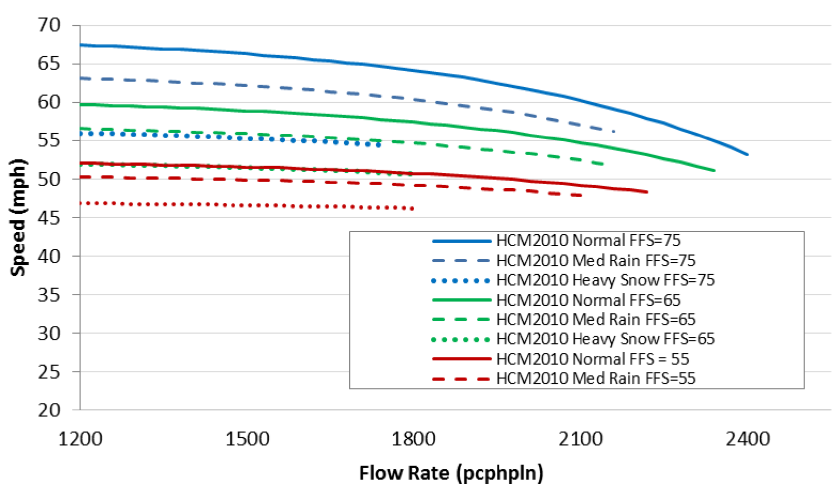


Figure 6.3. Example application of SAF and CAF for different base FFS and weather categories on merge (on-ramp) segments.

the speed calculation procedure for weave segments is modified to consider weather and incident reductions in free-flow speed, through the use of SAFs. The method separately estimates the speed of weaving and nonweaving vehicles, which are eventually combined to estimate a space mean speed of all vehicles in the segment. The equations for calculating the speed of weaving and nonweaving vehicles (Equations 12-19 and 12-20 in HCM2010) are modified by multiplying each occurrence of FFS by SAF (Equations 6.7, 6.8, and 6.9):

$$S_W = 15 + \left(\frac{\text{FFS} \times \text{SAF} - 15}{1 + W} \right) \tag{6.7}$$

$$W = 0.226 \left(\frac{\text{LC}_{\text{ALL}}}{L_S} \right)^{0.789} \tag{6.8}$$

$$S_{\text{NW}} = \text{FFS} \times \text{SAF} - (0.0072\text{LC}_{\text{MIN}}) - \left(0.0048 \frac{v}{N} \right) \tag{6.9}$$

In the next step, the space mean speed of all vehicles in the weaving segment is computed by HCM2010 Equation 12-20, repeated here as Equation 6.10:

$$S = \frac{v_W + v_{\text{NW}}}{\left(\frac{v_W}{S_W} \right) + \left(\frac{v_{\text{NW}}}{S_{\text{NW}}} \right)} \tag{6.10}$$

The variables used in Equations 6.7 through 6.10 are as follows:

- S_W = average speed of weaving vehicles within the weaving segment, mph;
- S_{NW} = average speed of nonweaving vehicles within the weaving segment, mph;
- FFS = free-flow speed of the weaving segment, mph;
- SAF = speed adjustment factor of the weaving segment;
- W = weaving intensity factor;
- L_S = length of the weaving segment, using the short length definition, ft (300 ft is the minimum value);
- LC_{ALL} = total lane-changing rate of all vehicles in the weaving segment, from HCM2010 Chapter 12, lane changes per hour;
- LC_{MIN} = minimum rate of lane changing that must exist for all weaving vehicles to successfully complete their weaving maneuvers, from HCM2010 Chapter 12 (lane changes per hour);
- v = total demand flow rate in the weaving segment = $v_W + v_{\text{NW}}$, pcph;
- v_W = weaving demand flow rate in the weaving segment, pcph;
- v_{NW} = nonweaving demand flow rate in the weaving segment, pcph;
- N = number of lanes within the weaving section; and
- S = space mean speed of all vehicles in the weaving segment.

Example Problem 1 from HCM2010 Chapter 12 was selected as the basis for a speed versus flow rate sensitivity analysis. The SAFs and CAFs used in this analysis are shown in Figure 6.4. Note that under these particular sets of inputs, speed varies linearly with flow. Also note that the figure is truncated for flow rates below 1,200 pcphpl.

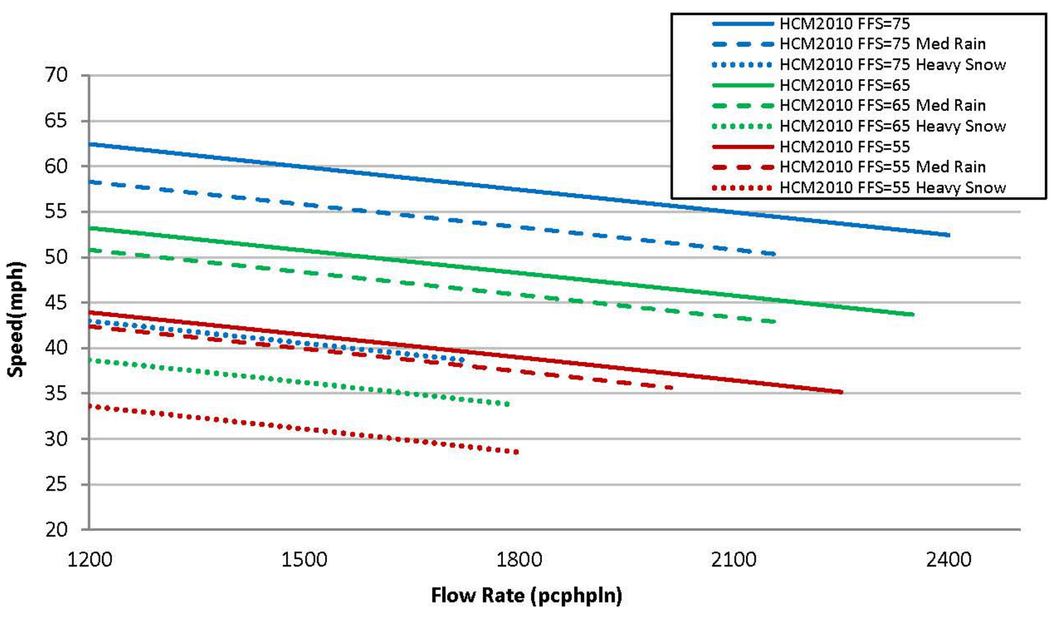


Figure 6.4. Example application of SAF and CAF for different base FFS and weather categories on weaving segments.

Table 6.4. Literature Synthesis of Appropriate SAFs and CAFs for Different Weather Conditions

Weather Type		Capacity Adjustment Factors (CAF)					Free-Flow Speed Adjustment Factors (SAF)				
Free-Flow Speed (mph)		55 mph	60 mph	65 mph	70 mph	75 mph	55 mph	60 mph	65 mph	70 mph	75 mph
Clear	Dry Pavement	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	Wet Pavement	0.99	0.98	0.98	0.97	0.97	0.97	0.96	0.96	0.95	0.94
Rain	≤0.10 in/h	0.99	0.98	0.98	0.97	0.97	0.97	0.96	0.96	0.95	0.94
	≤0.25 in/h	0.94	0.93	0.92	0.91	0.90	0.96	0.95	0.94	0.93	0.93
	>0.25 in/h	0.89	0.88	0.86	0.84	0.82	0.94	0.93	0.93	0.92	0.91
Snow	≤0.05 in/h	0.97	0.96	0.96	0.95	0.94	0.94	0.92	0.89	0.87	0.84
	≤0.10 in/h	0.95	0.94	0.92	0.90	0.88	0.92	0.90	0.88	0.86	0.83
	≤0.50 in/h	0.93	0.91	0.90	0.88	0.87	0.90	0.88	0.86	0.84	0.82
	>0.50 in/h	0.80	0.78	0.76	0.74	0.72	0.88	0.86	0.85	0.83	0.81
Temp	<50 deg F	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.98	0.98
	<34 deg F	0.99	0.99	0.99	0.98	0.98	0.99	0.98	0.98	0.98	0.97
	<−4 deg F	0.93	0.92	0.92	0.91	0.90	0.95	0.95	0.94	0.93	0.92
Wind	<10 mph	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	≤20 mph	0.99	0.99	0.99	0.99	0.99	0.99	0.98	0.98	0.97	0.96
	>20 mph	0.99	0.99	0.99	0.98	0.98	0.98	0.98	0.97	0.97	0.96
Visibility	<1 mi	0.90	0.90	0.90	0.90	0.90	0.96	0.95	0.94	0.94	0.93
	≤0.50 mi	0.88	0.88	0.88	0.88	0.88	0.95	0.94	0.93	0.92	0.91
	≤0.25 mi	0.90	0.90	0.90	0.90	0.90	0.95	0.94	0.93	0.92	0.91

New Defaults for CAF and SAF

The research team performed an extensive literature review on the impacts of incidents and weather events on both segment free-flow speed and capacity. Summaries are presented in Table 6.4 and Table 6.5. These tables show the new default values proposed for the HCM. Note that for incidents, the literature was inconclusive as to the effect on FFS, so a uniform SAF of 1.0 is assumed for incidents. An experienced analyst may choose to override these defaults with the support of local data or experience.

Enhanced Performance Measures for Congested Conditions

Because the focus of the L08 project is to incorporate nonrecurring congestion effects into the HCM2010, the procedure for

Table 6.5. Literature Synthesis of Appropriate CAFs for Different Incident Conditions

Number of Lanes (one direction)	No Incident	Shoulder Closure	One-Lane Closure	Two-Lane Closure	Three-Lane Closure	Four-Lane Closure
2	1.00	0.81	0.70	0.00	0.00	0.00
3	1.00	0.83	0.74	0.51	0.00	0.00
4	1.00	0.85	0.77	0.50	0.52	0.00
5	1.00	0.87	0.81	0.67	0.50	0.50
6	1.00	0.89	0.85	0.75	0.52	0.52
7	1.00	0.91	0.88	0.80	0.63	0.63
8	1.00	0.93	0.89	0.84	0.66	0.66

doing so is expected to be used to model highly oversaturated conditions. To facilitate such cases, new performance measures have been developed. These measures serve as additional checks of reasonableness for the analyst, and some are derived from the travel time distribution. The estimated travel time distribution is expected to be the most critical output from this methodology.

Denied Entry Queue Length

A new output variable added in FREEVAL-RL is the denied entry queue length (DEQL). The motivation for adding this variable was to identify severely congested scenarios for further analysis. Another advantage of calculating the DEQL is that the analyst gets a sense of the validity of the reliability performance measures. In other words, the HCM2010 methodology is not designed to handle all congested conditions; in particular, it has not been validated for very severe congestion scenarios. Therefore, the procedure may generate unrealistic results under those conditions. The DEQL can serve as a flag for these types of scenarios.

The DEQL informs the user of vehicle spillback out of the spatial domain of the coded facility in FREEVAL. Equation 6.11 is used to calculate DEQL at each analysis period inside the computational engine:

$$\text{Denied Entry Queue Length} = \frac{\text{UV}}{K_Q - K_B} \times 5{,}280 \qquad (6.11)$$

where

Denied entry queue length = denied entry queue length at the end of the analysis period, ft;

UV = number of unserved vehicles on the first segment of the facility at the end of the analysis period, veh;

K_Q = queue density, the vehicle density in the queue on the first segment of the facility at the end of the analysis period, veh/mi; calculated on the basis of a linear density–flow relationship in the congested regime inside the computational engine; and

K_B = background density, the first segment density over the analysis period assuming there is no queuing on the segment, veh/mi/lane; this density is calculated using the expected demand on the segment in the corresponding undersaturated procedure in Chapters 11 through 13 of the HCM2010.

Another advantage of representing DEQL is to give users a sense of how much they should expand the spatial scope of the coded facility. For example, the base scenario should preferably have no DEQL to ensure that the spatial extent of base congestion (no weather or incident effect) is fully contained within the facility. Similarly, the majority of scenarios should preferably result in zero or low denied entry queues, with only rare and very severe scenarios having higher queue estimates.

Travel Time Index for Entire Time-Space Domain

The travel times for each segment at each analysis period (time-space domain) are available as outputs in the original HCM2010 methodology. Therefore, the facility's travel time index (TTI) can simply be calculated by dividing individual travel times by the free-flow travel time. Equation 6.12 demonstrates this simple calculation:

$$\text{TTI}_{ij} = \text{TT}_{ij} / \text{FFTT}_i \qquad (6.12)$$

where

TTI_{ij} = travel time index on segment *i* in analysis period *j*;

TT_{ij} = travel time on segment *i* in analysis period *j*; and

FFTT_i = free-flow travel time on segment *i*.

Also, the facility TTI in each analysis period is calculated simply by dividing facility travel time at a specific time period by its free-flow travel time (Equation 6.13):

$$\text{TTI}_j = \text{TT}_j / \text{FFTT}_j \qquad (6.13)$$

where

TTI_j = facility travel time index in analysis period *j*;

TT_j = facility travel time in analysis period *j*; and

FFTT_j = facility free-flow travel time in analysis period *j*.

In applying the method to multiple scenarios, a separate TTI is generated for each 15-min analysis period in each scenario. These calculated TTIs, along with the corresponding probabilities produced by the freeway scenario generator, are used to develop a cumulative TTI distribution, as shown in Appendix A. The analyst may further decide to focus on the 50th, 85th, or 95th percentile TTI as a performance measure, as illustrated in Figure 6.5. The TTI distribution can further be segregated into recurring and nonrecurring scenarios, or it can be used to compare distributions based on different demand, weather, or incident conditions.

Automation of Computations

In order to evaluate multiple scenarios, some form of automation is required. The HCM freeway facilities method has long relied on the use of computational engines like FREEVAL-2010 to conduct the analysis. With the introduction of reliability analyses, FREEVAL needed to be adapted to run in batch mode. Essential information for reliability analysis is now saved from each run. Each run output is saved in a separate spreadsheet named according to the respective

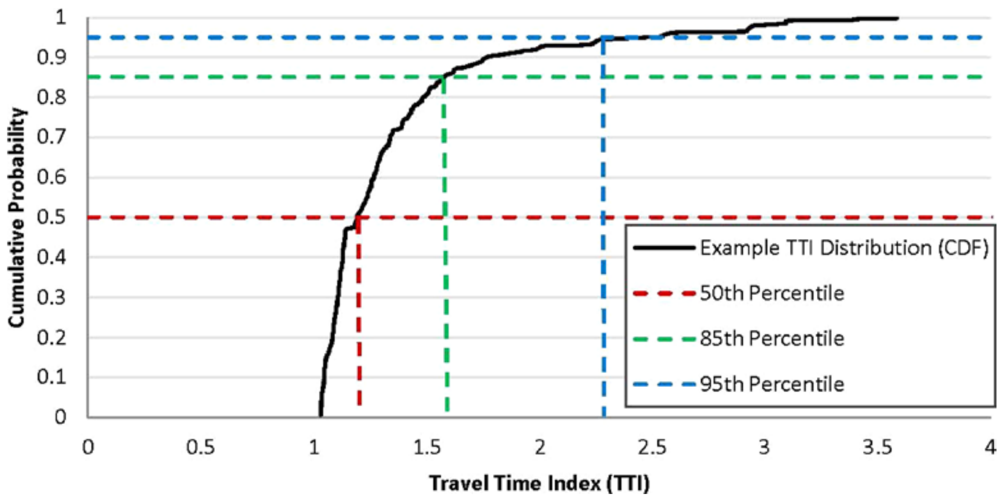


Figure 6.5. Sample cumulative TTI distribution with key percentiles. CDF = cumulative distribution function.

scenario. The saved output from each run can be categorized as follows:

- Scenario description;
- Analysis period detailed performance measures;
- Speed contour in the time–space domain; and
- Overall result summary sheet.

After the runs are completed, all scenario attributes are tabulated in a single spreadsheet for fast and efficient analysis. This summary report contains all the necessary information for each analysis period from all scenario runs. Each line represents a 15-min analysis period output for a given scenario.

FREEVAL-RL Calibration

Estimating the distribution of the travel time index (i.e., the ratio of average travel time to free-flow travel time) for a freeway facility involves using two computational engines. The first engine is the freeway scenario generator (FSG), which creates the different scenarios (unique combinations of demand patterns, weather conditions, incidents, work zones, and special events) that may be observed on a freeway facility, along with their individual probabilities. The second engine is FREEVAL-RL, which implements the HCM freeway facility methodology and calculates the travel time (and other performance measures) associated with each scenario.

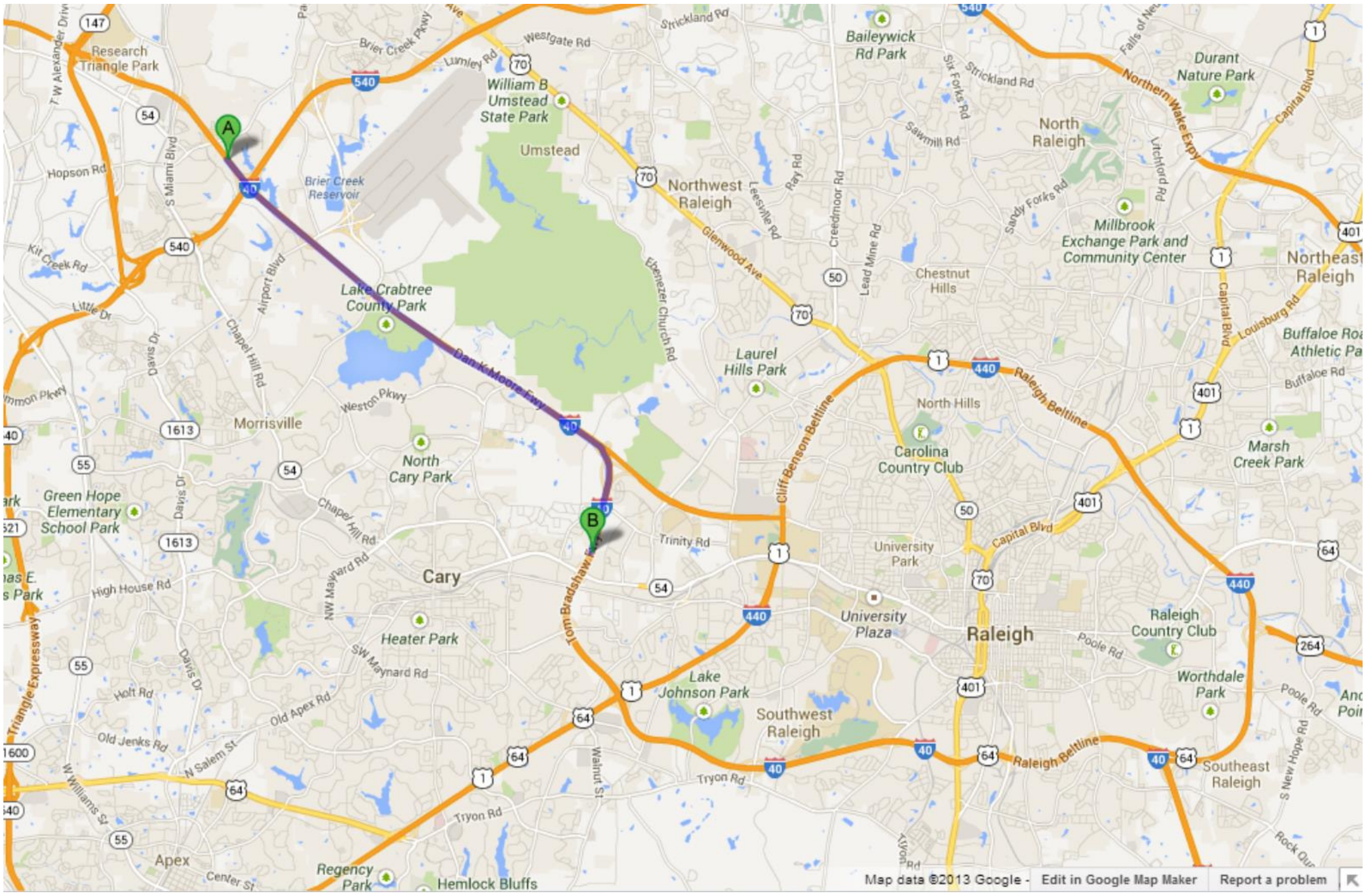
To fully calibrate the TTI distribution, several parameters in both the FSG and FREEVAL-RL can be adjusted to re-create observed operations in the field. This section describes the process of calibrating some of the key parameters available in FREEVAL-RL, without unduly complicating the calibration process. Traffic demand level is one of these parameters. Although traffic counts (or AADTs in data-poor environments) are used to calculate the entry traffic demand onto the facility, they are estimates of actual demand on the facility and can be significantly different from reality.

An incorrect estimate of the traffic demand is likely to lead to an inaccurate estimate of the facility travel time. This point makes clear the importance of calibrating the traffic demand and determining the level at which the resulting travel time distribution is as close as possible to the field-observed distribution. In addition, the values assumed for (1) the percent drop in capacity during traffic breakdown (α) and (2) jam density can yield significant changes in the travel time estimate resulting from changes in bottleneck throughput queue lengths and the speeds at which queues accumulate and dissipate, respectively.

Study Site and Data Sources

The calibration methodology was applied to a 12.5-mile freeway facility on eastbound Interstate 40 between mile markers 278.5 (point A in Figure 6.6) and 291.0 (point B) near Raleigh, North Carolina. The case study facility has a speed limit of 65 mph and a free-flow speed of 70 mph. The reliability reporting period (RRP) over which the analysis was carried out included all weekdays of calendar year 2010, and a study period from 2:00 to 8:00 p.m. The facility is primarily a commuter route that connects Durham to Raleigh, passing through the Research Triangle Park, a major employment center in the area. The two-way facility AADT was approximately 120,000 in 2010, and the eastbound facility experiences recurring congestion in the p.m. peak period.

Traffic demand data were estimated from counts extracted from permanent side-fire radar sensors located along the facility mainline. Temporary tube counters placed at the



Source: © 2013 Google.

Figure 6.6. I-40 facility location.

on- and off-ramps for a 2-week period were used to supplement the data because the ramps have no permanent sensors. Sidefire sensor data were collected for all of 2010 at the 15-min level; daily per-lane volumes were calculated at each sensor to determine combinations of days and months that operated similarly. Figure 6.7 shows trends in average daily traffic (ADT) per lane for 2010. Monday through Wednesday experience similar demand levels, while Thursday is more elevated and Friday has the highest demand. Although seasonal variation was not as significant, four seasons encompassing three months each (December–January–February; March–April–May; June–July–August; and September–October–November) were selected to group months with similar demands and similar weather conditions.

This process resulted in 12 separate demand groups, or patterns. Daily and monthly demand factors were calculated from the ratio of ADT for each combination of month and day for 2010 to the AADT. These values were then averaged for each of the 12 demand patterns emerging from the data. These patterns are depicted in Table 6.6 for each collection of contiguous cells with the same background color and border.

As part of the calibration, the overall demand levels were adjusted to determine the best demand level that recreates the observed operations. Fifteen-minute segment travel times were downloaded from the Regional Integrated Transportation Information System based on INRIX probe data that were collected across the facility during the RRP. The facility travel time was estimated from the segment travel times using a pseudotrajectory method based on the concept of "stitching" or "walking" the travel time. To identify typical operations with only recurring congestion effects, each 15-min period of the year was compared with weather and incident logs to confirm which time periods had no weather events or incidents.

Inclusion Thresholds

Theoretically, the reliability procedure can generate up to 22,932 detailed scenarios for the subject facility. Many of these may have exceptionally low or near-zero probability. In addition, some may be infeasible—for example, a two- or three-lane closure on a two-lane freeway segment. In this

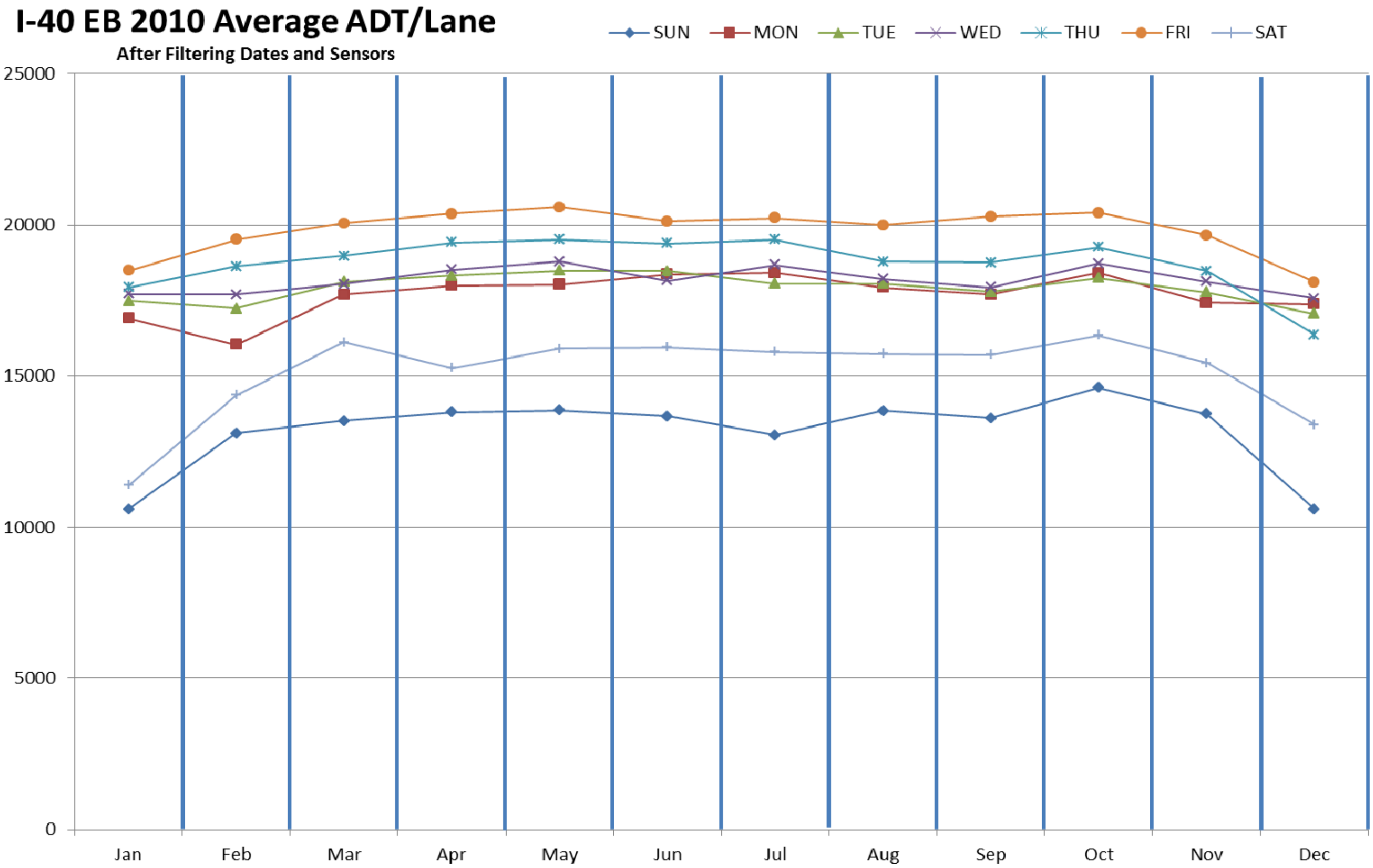


Figure 6.7. Facility average ADT per lane by month and day of the week.

Table 6.6. Demand Factors: Ratio of ADT to AADT by Month and Day of Week

Month	Sunday	Monday	Tuesday	Wednesday	Thursday	Friday	Saturday
January	0.617609	0.999005	1.030232	1.042881	1.055117	1.084198	0.662407
February	0.763747	0.941499	1.013144	1.041699	1.094640	1.142797	0.837179
March	0.794913	1.045799	1.071891	1.066066	1.113577	1.173921	0.940873
April	0.817347	1.076144	1.090055	1.100863	1.164751	1.217906	0.911421
May	0.815670	1.078904	1.108827	1.116618	1.160484	1.213328	0.933496
June	0.805796	1.080620	1.088449	1.070022	1.141443	1.183148	0.942226
July	0.764001	1.085168	1.073553	1.105148	1.150022	1.187813	0.933042
August	0.801063	1.048545	1.054661	1.062905	1.095856	1.167686	0.911527
September	0.768024	1.018452	1.026499	1.026072	1.077352	1.155702	0.893950
October	0.825240	1.051489	1.048223	1.069537	1.109691	1.163729	0.924886
November	0.756585	0.976373	1.002337	1.043700	1.084126	1.072912	0.829501
December	0.586780	0.977116	0.958762	0.989379	0.918297	1.010103	0.744283

case, the improbable and zero-probability detailed scenarios were removed from the reliability analysis. That translates to an inclusion threshold of near zero, meaning all scenarios with probability greater than zero were included in the analysis. Thus 2,058 scenarios were used in evaluating travel time reliability for the I-40 facility, as shown in Table 6.7.

In general, the scenarios with extremely low probability are not expected to be observed in the field in a single year; however, they are included in the predicted TTI distribution when an inclusion threshold of zero is used. As a result, a comparison of the predicted and observed distributions is hard to interpret: the predicted distributions include the low-probability scenarios, while the observed distribution may not include any of them. In addition, the low-probability scenarios tend to have exceptionally large TTI values that significantly shift the tail of the cumulative distribution to the right (i.e., toward higher TTI values). These scenarios may also result in demand shifts in the real world that are not directly accounted for in the freeway reliability method.

Therefore, the procedure allows the user to specify an inclusion threshold and include only scenarios with probability larger than a specified threshold. For instance, an inclusion threshold of 1.0% means that only the scenarios with probability larger than 0.01 are considered in the analysis. Figure 6.8 presents the TTI cumulative distributions for four different inclusion threshold values for the subject facility, as well as the observed TTI distribution obtained from the INRIX data warehouse. For the subject facility, including all the scenarios with a nonzero probability in the analysis (i.e., inclusion threshold = zero) resulted in a general overestimation in the TTI cumulative distribution. Increasing the threshold to 1.0% brought the TTI distribution much closer to the observed distribution. An inclusion threshold of 1.2% resulted in matching planning time index (PTI) values for the predicted and observed TTI distributions. Inclusion thresholds larger than 1.2% yielded a general underestimation in the TTI distribution.

Table 6.7. I-40 Facility: Final Scenario Categorization

Scenario Type	Number of Scenarios	Percent of Total
No incidents and nonsevere weather	12	0.6
No incidents and severe weather	66	3.2
Incidents and nonsevere weather	528	25.7
Incidents and severe weather	1,452	70.6
Total	2,058	100.0

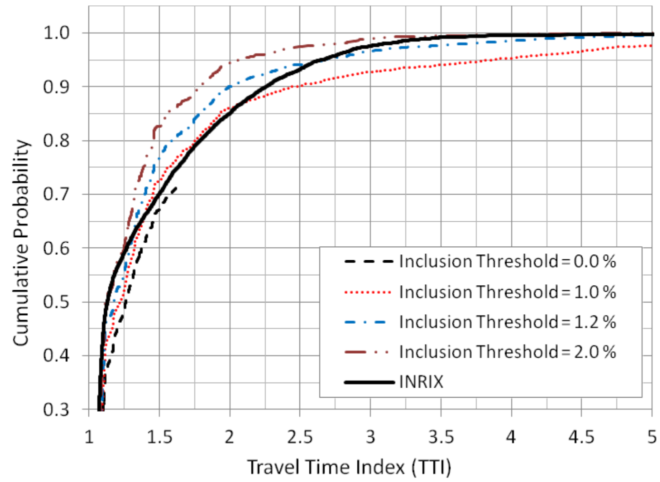


Figure 6.8. I-40 facility: Travel time distribution results for different inclusion thresholds.

Increasing the value of the inclusion threshold reduces the number of scenarios and consequently the computational engine run time; however, at the same time, it reduces the percentage of the coverage of feasible scenarios (Table 6.8). In other words, the larger the value of the inclusion threshold, then the greater the number of scenarios excluded from the analysis. As a result, fewer feasible scenarios are covered in the analysis.

As shown in Table 6.8, the number of scenarios significantly drops as the value of the inclusion threshold is increased. Going from an inclusion threshold of 0.00% to 0.01% eliminates half of the scenarios and decreases the coverage of the distribution by only 0.29%. This means that more than 1,000 of the scenarios contributed to only 0.29% of the TTI distribution.

Table 6.8. I-40 Facility: Number of Scenarios and Coverage of Feasible Scenarios

Inclusion Threshold (%)	Number of Scenarios	Coverage of the Distribution (%)
0.00	2,058	100.00
0.01	1,004	99.71
0.10	496	97.46
1.00	264	89.63
1.20	210	85.07
1.30	174	82.55
2.00	84	75.91
3.00	81	67.04
4.00	4	37.32

Summary of Freeway Model Enhancements

The enhancements to the FREEVAL-RL computational engine include the following:

- Incorporating the two-capacity phenomenon under queue discharge conditions;
- Incorporating SAFs for certain nonrecurring congestion sources;
- Improving modeling of CAFs and SAFs for merge, diverge, and weaving segments;
- Adding new defaults for CAFs and SAFs for incidents and weather events on freeways;
- Extending performance measures for congested conditions; and
- Automating computation.

The output of the enhanced computational engine is consistent with the HCM.

Moreover, this section documents the calibration process for generating a cumulative TTI distribution for freeway travel time reliability purposes (to be applied before incorporating any weather or incident effects), using the expanded HCM2010 approach to estimate facility reliability. In this process, three calibration parameters were tested: the traffic demand-level adjustment, the percent capacity drop during breakdown (α), and the facility jam density. Three values for each calibration parameter were evaluated, resulting in a total of 27 parameter combinations. Cumulative TTI distributions for each parameter combination were compared with the observed cumulative TTI generated from INRIX travel time data. The distributions were generated for a 12.5-mile freeway facility (eastbound I-40 near Raleigh, North Carolina).

The statistical analysis revealed that increasing the overall base demand level in the seed file by 3.0% and using a value of 9% for α resulted in cumulative TTI distributions that were not statistically different from the observed cumulative TTI distributions. This conclusion applied to all jam density values because the results indicated that the estimated distribution was not sensitive to jam density over the range of parameter values investigated. In addition, increasing the traffic demand adjustment factor and the breakdown capacity reduction factor (α) resulted in increased TTI values and a shift in the TTI distribution toward the right, as expected. The large difference in some TTI distributions between HCM2010 and INRIX at higher percentiles could be attributed to unreported events that may have affected demand and capacity over the course of the year. The travel time effects of unreported events would still be present in the INRIX data set.

Urban Streets Enhancements

This section describes the enhancements made to the urban street segment methodology described in Chapter 17 of the HCM2010. The methodology is used to evaluate the operation of undersaturated street segments. The enhancements described in this section extend the HCM2010 methodology to the evaluation of the operation of urban street facilities with one or more oversaturated segments.

Three enhancements are described in this section. The first is a procedure for adjusting the discharge rate from a signalized intersection when a downstream incident or work zone blocks one or more lanes on the segment. The second is a procedure for computing the effective average vehicle spacing on a segment with spillback. The third is a process for using the HCM2010 methodology to evaluate urban street facilities with spillback in one or both travel directions on one or more segments.

Mid-Segment Lane Restriction

When one or more lanes on an urban street segment are closed, the flow in the lanes remaining open will be adversely affected. Occasionally, this blockage can have an adverse effect on the performance of movements entering the segment at the upstream signalized intersection and on those exiting the segment at the downstream signalized intersection. The nature of these impacts is shown in Figure 6.9 for a work-zone-related lane blockage. The impacts are similar for mid-segment incidents.

In Figure 6.9, the mid-segment work zone is shown to influence the saturation flow rate of movements at both the upstream and downstream signalized intersections. Logically, the magnitude of the effect will increase as the distance between the intersection and work zone decreases. The mid-segment work zone can also influence segment travel time, especially if the demand exceeds the work zone capacity during a portion (or all) of the signal cycle. These influences are shown for one direction of travel; however, the work zone can be located in the middle of the street such that it influences both directions of travel.

Three areas of impact are identified in Figure 6.9. The effect on the upstream intersection saturation flow rate is the subject of discussion in this section. The effect on segment travel time and capacity is addressed in the Chapter 5 section Urban Street Scenario Development. The effect on the downstream intersection saturation flow rate is described in Appendix J.

Procedure

The methodology described in HCM2010 Chapter 17 is shown in the flowchart in Figure 6.10. It consists of five main

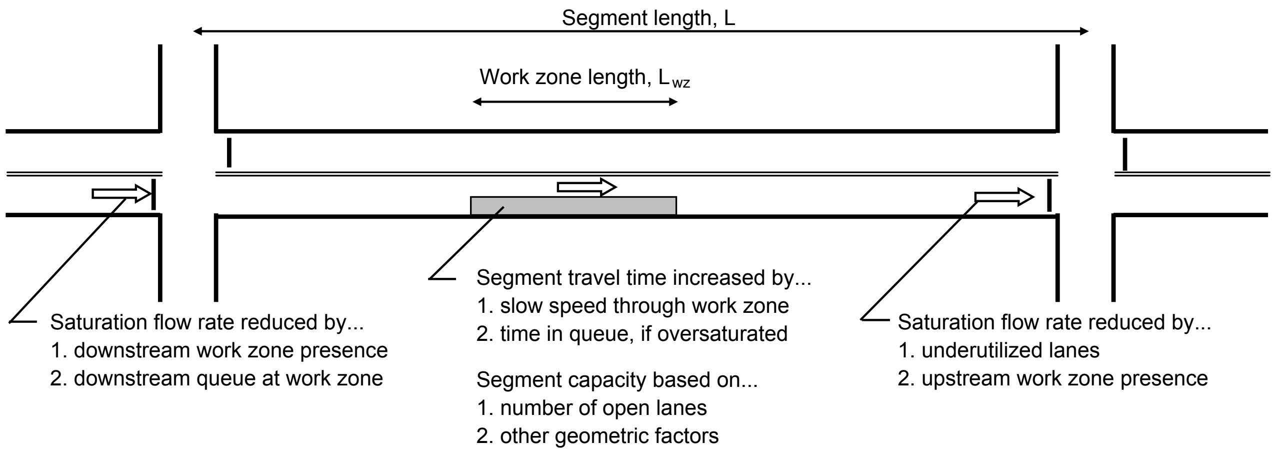


Figure 6.9. Mid-segment work zone impacts.

modules that are completed in sequence to produce a reliable estimate of street segment performance. In application, the methodology is repeated for each segment of the facility. The results of each application are aggregated to produce an estimate of facility performance. The HCM2010 provides more detail about these modules.

The procedure described in this section is used to adjust the saturation flow rate of the movements entering a segment when one or more downstream lanes are blocked. The procedure was developed for incorporation within the HCM Chapter 17 methodology, specifically, the segment evaluation module. The sequence of calculations in the segment

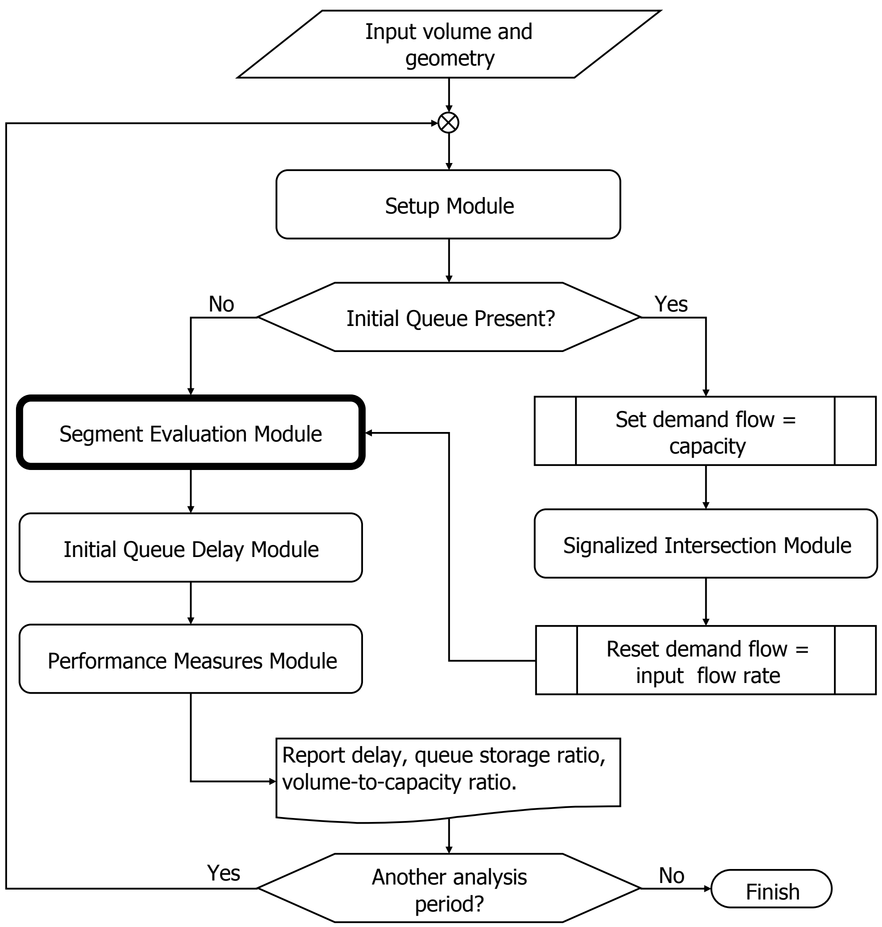


Figure 6.10. Methodology flowchart.

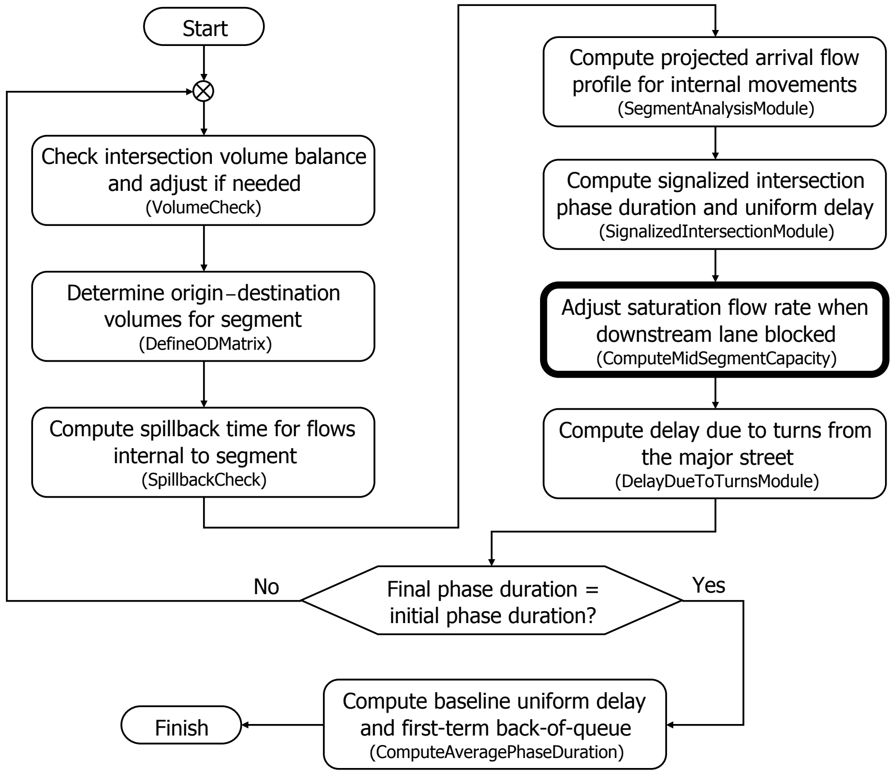


Figure 6.11. Segment evaluation methodology.

evaluation module is shown in Figure 6.11. The module comprises eight procedures. As shown in the figure, the module is implemented in an iterative loop which repeats until convergence on the estimated phase duration is achieved.

The relevant procedure is implemented in the sixth computational routine, "ComputeMidSegmentCapacity," outlined by a thick bold line. It compares the estimate of movement capacity (computed in the previous procedure) with the downstream lane capacity. If the movement capacity exceeds the downstream lane capacity, then the movement saturation flow rate is reduced accordingly. This can occur when one or more downstream lanes are blocked because of a work zone or an incident.

A new saturation flow rate adjustment factor is introduced by the procedure. This factor is computed for each movement entering the subject segment. Equations 6.14 and 6.15 are used to compute the factor value:

$$\text{If } c_{ms} < c_i \text{ or } f_{ms,i-1} < 1.0 \text{ then: } f_{ms,i} = f_{ms,i-1} \times \frac{c_{ms}}{c_i} \geq 0.1$$

$$\text{otherwise: } f_{ms,i} = 1.0 \qquad (6.14)$$

with

$$c_{ms} = 0.25\, k_j\; N\, S_f \leq 1{,}800\; N \qquad (6.15)$$

where

$f_{ms,I}$ = adjustment factor for downstream lane blockage during iteration i;
c_{ms} = mid-segment capacity, veh/h;
c_i = movement capacity during iteration i, veh/h;
k_j = jam density (= 5,280/L_h), veh/mi/lane;
L_h = average vehicle spacing in stationary queue, ft/veh;
S_f = free-flow speed, mph; and
N = number of lanes.

The number of lanes used in Equation 6.15 equals the number of unblocked lanes (i.e., the open lanes) while the blockage is present.

The variable i in the adjustment factor subscript indicates that its value is incrementally revised with each subsequent iteration. Ultimately, it converges to a value that results in a movement capacity that matches the available mid-segment capacity. For the first iteration, the factor value is set to 1.0 for all movements. The factor value is also set to 1.0 if the segment is experiencing spillback. In that situation, a saturation flow rate adjustment factor for spillback (which incorporates the downstream lane blockage effect) is computed for the movement. The calculation of the factor for spillback is described in a subsequent subsection.

Equation 6.15 indicates that the factor is less than 1.0 when the mid-segment capacity is smaller than the movement capacity. If the factor has been set to a value less than 1.0 in a previous iteration, then the factor continues to be adjusted with each subsequent iteration until convergence is achieved. A minimum factor value of 0.1 is imposed as a practical lower limit.

Equation 6.15 is based on the linear speed-density relationship developed by Greenshields (1934) and the fundamental relationships among flow, speed, and density. These relationships underlie the vehicle-proximity adjustment factor used in the HCM2010 Chapter 17 methodology (i.e., Equation 17-5) to compute segment running speed. When the average vehicle spacing L_h is 25 ft/veh, the mid-segment capacity is computed as $c_m = 52.8\ N\ S_f$.

The saturation flow rate adjustment factor for downstream lane blockage is applicable to all signalized intersection movements that enter the urban street segment of interest. It is used to adjust the saturation flow rate of these movements. If implemented in the HCM2010, it would be added to Equation 18-5 in Chapter 18. It would also be multiplied by the result obtained from Equations 31-59, 31-61, 31-62, 31-63, 31-101, 31-102, 31-104, 31-105, 31-106, 31-107, and 31-116 in HCM2010 Chapter 31.

For those entry movements that have permissive or protected-permissive left-turn operation, the adjustment factor is also used to adjust the number of left-turn sneakers per cycle. This adjustment is shown in Equation 6.16:

$$\text{If exclusive left-turn lane then: } n_{s,a} = n_s\ f_{\text{ms}}$$
$$\text{If shared left-turn lane then: } n_{s,a} = (1 + P_L)\ f_{\text{ms}} \tag{6.16}$$

where

$n_{s,a}$ = adjusted number of sneakers per cycle (= 2.0), veh;
n_s = number of sneakers per cycle, veh; and
P_L = proportion of left-turning vehicles in the shared lane.

The change suggested by Equation 6.16 requires multiplying the factor f_{ms} by the result obtained from Equation 31-60 in Chapter 31 of the HCM2010. This factor should also be multiplied by the n_s term in Equations 31-113, 31-118, and 31-119, and by the $(1 + P_L)$ term in Equations 31-115 and 31-120.

Effective Average Vehicle Spacing

When an urban street segment experiences spillback, traffic movements at the upstream signalized intersection will be severely limited in the ability to serve traffic demand. Specifically, the upstream movements that are destined for entry into the segment may be blocked by queued vehicles for some or all of the green indication (green traffic light). Thus, spillback effectively reduces the capacity of these movements.

Segment spillback falls into two categories. One type is called sustained spillback. It represents a condition in which the volume entering the segment exceeds the capacity of the downstream intersection for sufficient time to allow queued vehicles to extend for the length of the segment. Sustained spillback is a consequence of inadequate capacity. The period of sustained spillback starts the first time that vehicles stop on the segment because of the downstream signal and then block (or slow) the departure of one or more upstream movements desiring to enter the segment.

A second type of segment spillback is called cyclic spillback. It represents a condition in which the volume entering the segment does *not* exceed the capacity of the downstream intersection, but the signal timing (i.e., phase duration and offset) relationship between the upstream and downstream intersections is such that a queue of stopped vehicles can extend for the length of the segment for a portion of the signal cycle. Random cycle-to-cycle variation in demand and capacity can increase the frequency and extent of this type of spillback. Cyclic spillback is more likely to occur at signalized interchanges and closely spaced signalized intersections.

The remainder of the discussion in this subsection addresses sustained spillback because it is associated with large delays caused by congested conditions. Note that the interchange ramp terminals methodology in Chapter 22 of the HCM2010 addresses cyclic spillback.

Chapter 30 of the HCM2010 describes a procedure for computing the time that spillback occurs on a segment, relative to the start of a specified analysis period (and given any initial queue present at that time). One step in this procedure requires the calculation of the maximum queue storage on the segment. This calculation is based on the average vehicle spacing in a stationary queue L_h. Specifically, the maximum queue storage value is computed by dividing the length of segment available for storage by L_h. This calculation can overestimate the actual number of queued vehicles needed to precipitate segment spillback. The bias stems from the assumption that all vehicles on the segment will always be stationary when spillback occurs. This is a weak assumption because the downstream signal operation creates backward-traveling waves of starting and stopping. Between the starting wave and the stopping wave, vehicles are moving at the saturation headway and its associated speed. This behavior is illustrated in Figure 6.12.

Figure 6.12 illustrates the position of vehicles during one point in time on a segment with spillback. Specifically, it indicates vehicle positions a few seconds after the onset of the red signal indication. The first four vehicles are shown to be stopped in the queue. The next five vehicles are moving at the saturation headway. The remaining vehicles are shown to be stopped. Those remaining vehicles will begin moving forward in a few seconds. The point of this figure is that the maximum queue storage value is less than that computed using the HCM2010 method because the spacing of the moving vehicles is larger

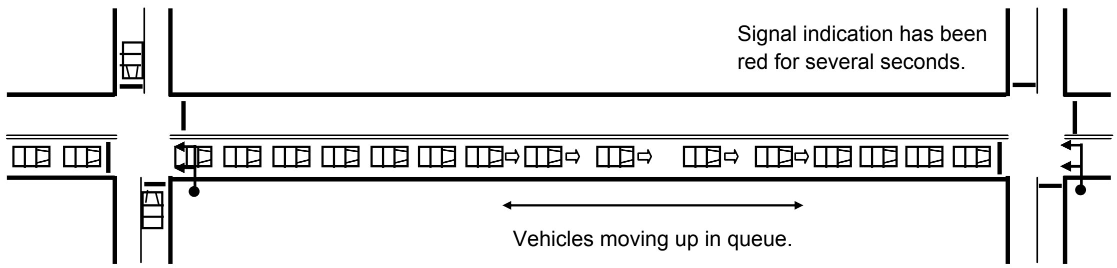


Figure 6.12. Vehicle position seconds after onset of red indication.

than L_h. This observation will always be true when the segment length is sufficiently long that the stopping wave does not reach the upstream signal before the onset of the next green indication.

The procedure described in this section is used to estimate the effective average vehicle spacing (L_h^*) on a segment with spillback. The derivation of this new variable is based on the vehicle trajectories shown in Figure 6.13. The segment of interest is shown on the left side of the figure. Spillback is present for all of the cycles shown; however, trajectories are shown for only two cycles. The solid trajectories coincide with vehicles that enter the segment as a through movement at the upstream intersection. The dashed lines coincide with vehicles that enter the segment as a turn movement. A vehicle that enters the segment traveling north as a through vehicle is shown to experience four cycles before exiting the segment. The trajectories show that the vehicles move forward at a saturation headway of 3,600/s seconds per vehicle (where s is the saturation flow rate in vehicles per hour) and a speed of V_a ft/s.

The lines that slope downward from the upper left to lower right represent the waves of reaction time. They have a slope of t_{pr} seconds per vehicle. The starting wave originates at the onset of the green indication and the stopping wave originates at the onset of the red indication. The average vehicle spacing when vehicles are stopped is L_h feet per vehicle.

The relationship between the trajectories of the moving vehicles in Figure 6.13 defines the following relationship between speed, saturation headway, vehicle length, and driver starting response time t_{pr} (Equation 6.17).

$$t_{pr} = \frac{3{,}600}{s} - \frac{L_h}{V_a} \tag{6.17}$$

where

t_{pr} = driver starting response time, s/veh; and
s = saturation flow rate, veh/h;
L_h = average vehicle spacing in stationary queue, ft/veh; and
V_a = average speed of moving queue, ft/s.

Driver starting response time and the distance between vehicles in a stopped queue at signalized intersections have been the subject of several previous studies. Messer and Fambro (1977) found that driver response was fairly constant at 1.0 s, regardless of queue position. The only exception was the driver in the first queue position who had an additional

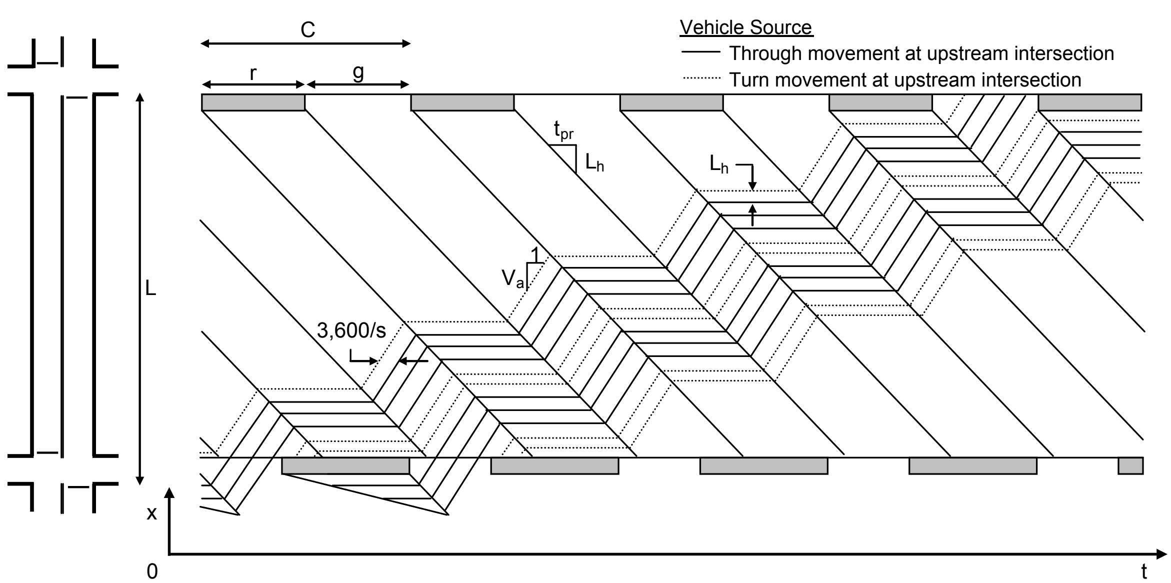


Figure 6.13. Vehicle trajectories during spillback conditions.

delay of 2.0 s. The shorter response time of the second and subsequent queued drivers is likely due to their ability to anticipate the time to initiate motion by seeing the signal change and/or the movement of vehicles ahead. Messer and Fambro also found that the average length of roadway occupied by each queue position is about 25 ft.

Another study of driver response time was conducted by George and Heroy (1966). They found driver response to be relatively constant at about 1.3 s for all queue positions. However, further examination of their data suggests that the first driver's response time was slightly longer, at about 1.5 s to 2.0 s.

Response times in the preceding studies were all measured at the start of vehicle motion. A study found that driver response to disturbance (including the start of motion) remained fairly constant as the platoon of queued vehicles increased its speed (Herman et al. 1971). In particular, they found that the speed of propagation of the response wave was relatively constant at about 26 ft/s up to platoon speeds of 30 ft/s. By using an average distance between stopped vehicles of 25 ft, the starting response time for this wave speed can be calculated as 1.0 s (= 25/26).

Bonneson (1992) evaluated discharge headway data by using a regression model based on Equation 6.17. He found that a starting response value of 1.34 s/veh provided the best fit to headway data at signalized intersections.

On the basis of the relationships shown in Figure 6.13, the following procedure can be used to estimate the effective average vehicle spacing.

Step 1: Compute Wave Travel Time

The time required for the driver reaction wave to propagate backward to the upstream intersection is computed using Equation 6.18:

$$t_{\max} = \frac{L_{a,\text{thru}} \times t_{\text{pr}}}{L_h} \tag{6.18}$$

where

$t_{\max}$ = wave travel time, s;
$L_{a,\text{thru}}$ = available queue storage distance for the through movement, ft; and
t_{pr} = driver starting response time (= 1.3), s/veh.

The available queue storage distance for the through movement $L_{a,\text{thru}}$ equals the segment length less the width of the upstream intersection.

A value of 1.3 seconds per vehicle is recommended for the driver starting response time t_{pr}. This value is based on the findings from past research summarized in the previous subsection.

The average vehicle spacing in a stationary queue can be estimated using Equation 31-149 from Chapter 31 of the HCM2010. This equation estimates spacing for traffic streams composed of passenger cars and trucks. The discussion in Chapter 31 indicates that a value of 25 ft/veh can be used for the average spacing of passenger-car-only traffic streams.

Step 2: Compute Speed of Moving Queue

The average speed of the moving queue is computed using Equation 6.19. This equation was derived from Equation 6.17.

$$V_a = \frac{L_h}{(3{,}600/s) - t_{\text{pr}}} \tag{6.19}$$

When the average vehicle spacing is 25 ft/veh, the saturation flow rate is 1,800 veh/h, and the driver starting response time is 1.3 s/veh, then the average speed of the moving queue is computed as 35.7 ft/s.

Step 3: Compute Effective Average Vehicle Spacing

The relationship between the trajectories of the moving vehicles defines the following relationships among speed, saturation flow rate, signal timing, and vehicle spacing (Equation 6.20):

$$\text{If } 0.0 \le t_{\max} < r \text{ then:} \quad L_h^* = L_h$$

$$\text{If } r \le t_{\max} < C \text{ then:} \quad L_h^* = 3{,}600\left(\frac{rs}{L_{a,\text{thru}}} + \frac{s}{V_a}\right)^{-1}$$

$$\text{If } r \le t_{\max} < C \text{ then:} \quad L_h^* = \frac{L_h}{1.0 - t_{\text{pr}}(g/C)(s/3{,}600)} \tag{6.20}$$

where

L_h^* = effective average vehicle spacing in stationary queue, ft/veh;
r = effective red time (= $C - g$), s;
g = effective green time, s; and
C = cycle length, s.

Equation 6.20 has three component equations. Which component equation is used for a given segment and analysis period depends on the values of $t_{\max}$, r, and C. The value of average vehicle spacing from the first component equation represents the smallest value that can be obtained from Equation 6.20. The value from the last component equation represents the largest value that can be obtained. The value obtained from the middle component equation varies between those two extreme values, depending on the value of $t_{\max}$.

The procedure described in this section is used to estimate the effective average vehicle spacing L_h^* on a segment with spillback. This estimate is intended for use with the spillback check procedure documented in Chapter 30 of HCM2010. The spillback check procedure is used to estimate the time until spillback. The variable L_h^* should be substituted for L_h in

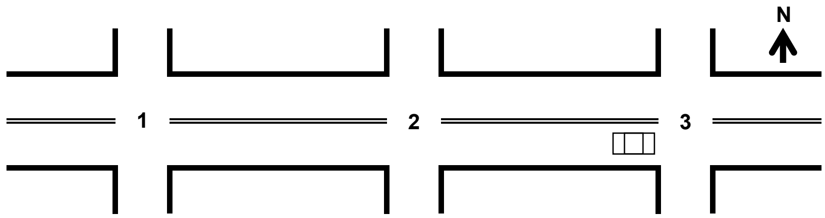


Figure 6.14. Example facility at start of analysis period.

Chapter 30. The result will be a more reliable estimate of the time until spillback.

Sustained Spillback

This subsection describes a methodology for using the HCM2010 urban streets methodology to evaluate a facility with spillback in one or more travel directions on one or more segments. This discussion addresses sustained spillback, as already defined.

The effect of spillback on traffic flow is modeled through an iterative process that repeatedly applies the HCM2010 methodology to the subject urban street facility. If spillback occurs on a segment, then the discharge rate of the traffic movements entering the segment are reduced such that (1) the number of vehicles entering the segment equals the number of vehicles exiting the segment and (2) the residual queue length equals the available queue storage distance.

A conceptual overview of the spillback methodology follows. The approach used to model spillback effects is similar to the multiple-time-period analysis procedure described in Chapter 18 of HCM2010. However, in this application, a single analysis period is divided into subperiods for separate evaluation. Each subperiod is defined using the following rules:

- The first subperiod starts with the start of the analysis period.
- The current subperiod ends (and a new subperiod starts) with each new occurrence of spillback on the facility.
- The total of all subperiod durations must equal the original analysis period duration.

As with the multiple-time-period analysis procedure, the residual queue from one subperiod becomes the initial queue for the next subperiod. When all subperiods have been evaluated using the HCM2010 methodology, the performance measures for each subperiod are aggregated for the analysis period using a weighted-average technique, in which the weight is the volume associated with the subperiod.

The spillback modeling approach is described by applying it to a simple two-segment urban street facility. It supports travel in both directions, so four occurrences of spillback are possible. The analysis period is 0.25 h, which coincides with the time interval 0.0 h to 0.25 h. The facility is shown in Figure 6.14 along with the initial queue at the start of the analysis period.

The HCM2010 methodology is used to evaluate the facility. The results indicate that spillback occurs on segment 2–3 in the eastbound travel direction (i.e., EB 2-3). This condition is shown in Figure 6.15. The time until spillback is 0.10 h, which is before the end of the analysis period. As a result, the predicted travel time for the eastbound direction is not correct. Therefore, additional evaluation is needed.

The HCM2010 methodology is used again to evaluate the facility, but this time the analysis period is reduced to 0.10 h, which coincides with the time interval 0.0 h to 0.10 h. The initial queue for EB 2-3 is still 1.0 vehicle. The results from the evaluation again indicate that the time until spillback is 0.10 h. However, the predicted travel time for this subperiod is correct because the time until spillback does not exceed the analysis period.

At this point, the results reflect only the time period 0.0 h to 0.10 h. Additional evaluation is needed to estimate the facility performance for the time period 0.10 h to 0.25 h.

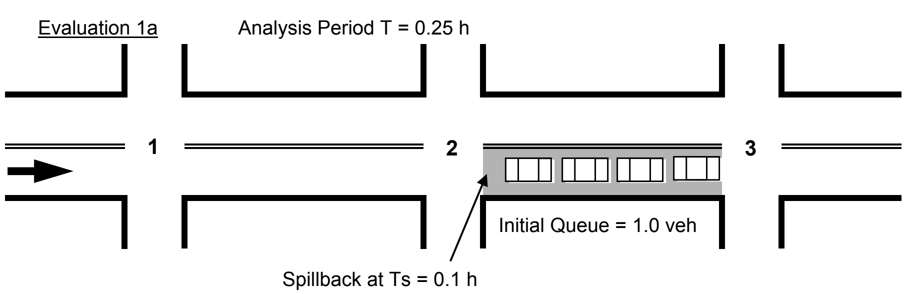


Figure 6.15. Example facility at time 0.10 hours.

Therefore, the HCM2010 methodology is again used to evaluate the facility. The analysis period is 0.15 h (= 0.25 − 0.10), which coincides with the time interval 0.10 h to 0.25 h. The facility is shown in Figure 6.16. For this subperiod, the initial queue for EB 2-3 is 4.0 vehicles. Also, the saturation flow rate of each movement entering EB 2-3 is reduced to ensure that the residual queue on EB 2-3 does not exceed the available queue storage distance in subsequent time intervals.

The results indicate that spillback occurs on segment 1–2 in the eastbound travel direction (i.e., EB 1-2). This condition is shown in Figure 6.16. The time until spillback is 0.05 h, which is before the end of the analysis period. As a result, the predicted travel time for the eastbound direction is not correct. Once again, additional evaluation is needed.

The HCM2010 methodology is used to evaluate the facility, but this time the analysis period is reduced to 0.05 h, which coincides with the time interval 0.10 h to 0.15 h. The initial queue for EB 2-3 is still 4.0 vehicles. The results from the evaluation again indicate that the time until spillback is 0.05 h. However, the predicted travel time for this subperiod is correct because the time until spillback does not exceed the analysis period.

At this point, the results reflect only the time periods 0.0 h to 0.10 h and 0.10 h to 0.15 h. Additional evaluation is needed to estimate the facility performance for the time period 0.15 h to 0.25 h. The HCM2010 methodology is again used to evaluate the facility. The analysis period is 0.10 h (= 0.25 − 0.15), which coincides with the time interval 0.15 h to 0.25 h. The facility is shown in Figure 6.16. For this subperiod, the initial queue for EB 2-3 is 4.0 vehicles and that for EB 1-2 is also 4.0 vehicles. The saturation flow rate of each movement entering EB 1-2 and EB 2-3 is reduced to ensure that the residual queue on EB 1-2 and on EB 2-3 does not exceed the available queue storage distance in subsequent time intervals.

The results of this evaluation indicate that no new spillback occurs. So, the predicted travel time for this subperiod is correct. The average travel time for the facility is computed as a weighted-average travel time for each of the three subperiods, in which the weight used is the subperiod volume.

The sequence of calculations in the spillback methodology is shown in Figure 6.17. It consists of several routines and two loops, one of which is an iterative loop with a convergence criterion. The HCM2010 urban streets methodology is implemented at three separate points in the flowchart.

Following the logic flow from the Start box, the HCM2010 methodology is initially implemented and the presence of spillback is checked. If spillback does not occur, then the results are reported and the process is concluded. If spillback occurs on a segment, then a subperiod is defined and the HCM2010 methodology is reimplemented using an analysis period that is shortened to equal the time until spillback.

The iterative loop shown on the right side of Figure 6.17 is called to quantify a saturation flow rate adjustment factor for each movement entering the segment with spillback. The value of this factor is determined to be the value needed to limit the entry movement volume such that the residual queue on the segment does not exceed the available queue storage distance.

The following subsections describe the spillback methodology as a sequence of computational steps that culminate in the calculation of facility performance for a specified analysis period. The input data requirements for this methodology are the same as for the HCM2010 urban streets methodology.

Step 1: Initialize Variables

Set the original analysis period variable T_o equal to the analysis period T input by the analyst. Set the total time variable $T_{\text{total},0}$ equal to zero and the subperiod counter k to 0.

Step 2: Implement the HCM2010 Methodology

The HCM2010 methodology is implemented in this step to evaluate the facility described by the input data. The analysis period duration is computed as $T = T_o - T_{\text{total},k}$. Increase the value of the subperiod counter k by 1.0.

Step 3: Check for Spillback

During this step, the results from Step 2 are examined to determine if new spillback has occurred. One direction of travel on

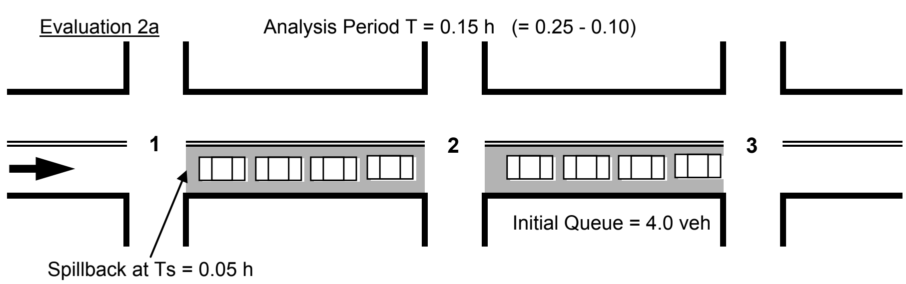


Figure 6.16. Example facility at time 0.15 hours.

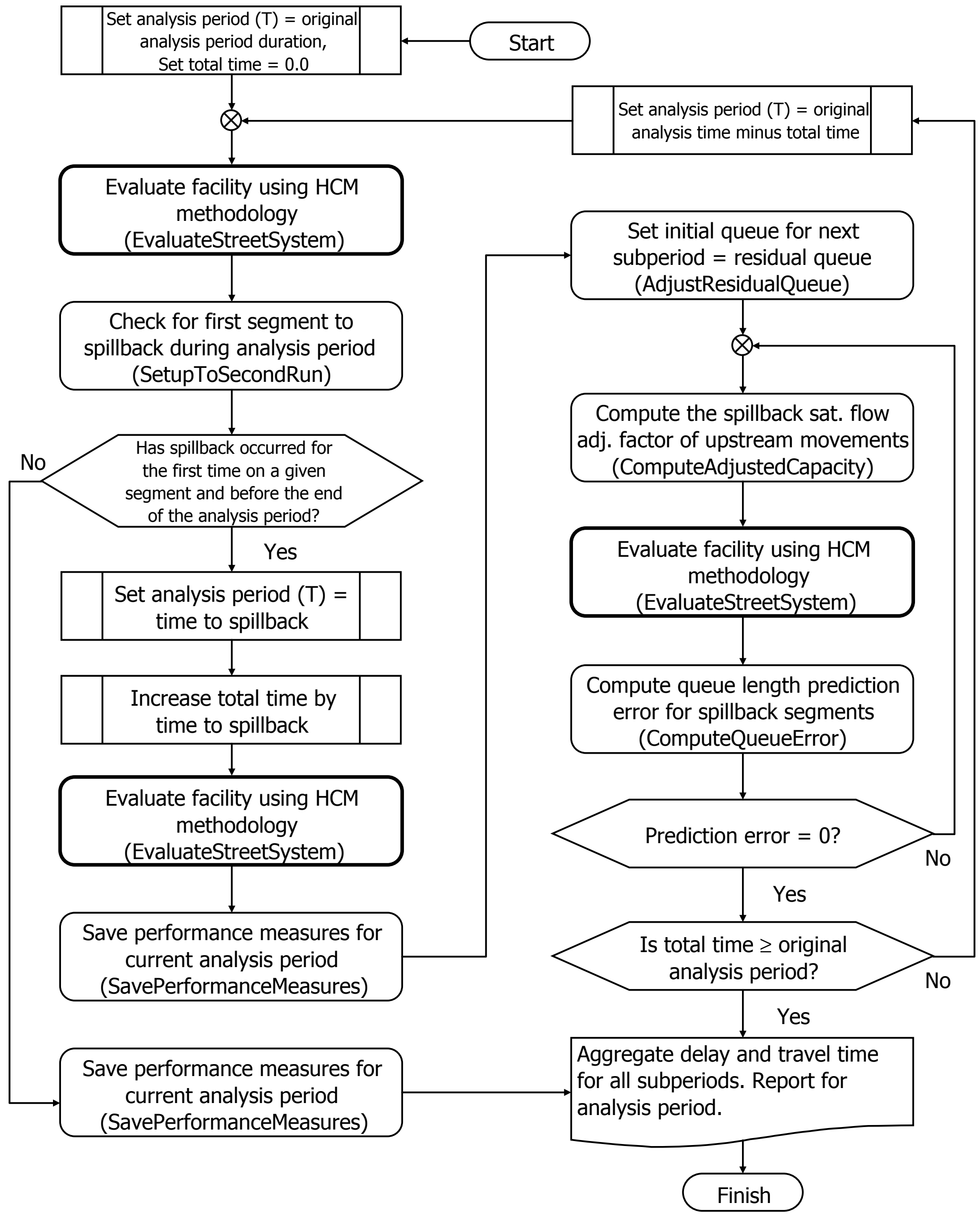


Figure 6.17. Spillback methodology flowchart.

one segment is considered a *site.* Each site is checked in this step. Any site that has experienced spillback during a previous subperiod is not considered in this step.

The predicted controlling time until spillback is recorded in this step. If several sites experience spillback, then the time of spillback that is recorded is based on the site experiencing spillback first. The site that experiences spillback first is flagged as having spilled back. The controlling time until spillback for the subperiod $T_{cs,k}$ is set equal to the time until spillback for this site. The total time variable is computed using Equation 6.21, which represents a cumulative total time for the current, and all previous, subperiods.

$$T_{total,k} = T_{total,k-1} + T_{cs,k} \tag{6.21}$$

where $T_{total,k}$ is equal to the total analysis time for subperiods 0 to k, in hours, and $T_{cs,k}$ is equal to the controlling time until spillback for the subperiod k, in hours.

If spillback does not occur, then the performance measures from Step 2 are saved using the procedure described in a subsequent subsection. The analyst then proceeds to Step 10 to determine the aggregate performance measures for the analysis period.

Step 4: Implement the HCM2010 Methodology to Evaluate a Subperiod

At the start of this step, the analysis period is set equal to the controlling time determined in Step 3 (i.e., $T = T_{cs,k}$). All other input variables remain unchanged. Then, the HCM2010 methodology is implemented to evaluate the facility. The performance measures from this evaluation are saved using the procedure described in a subsequent subsection.

Step 5: Prepare for the Next Subperiod by Determining the Initial Queue

During this step, the input data are modified by updating the initial queue values for all movement groups at each intersection. This modification is necessary to prepare for a new evaluation of the facility for the next subperiod. The initial queue for each movement group is set to the estimated residual queue from the previous evaluation.

The initial queue values for the movement groups at the downstream intersection that exit each segment are checked by comparing them with the available queue storage distance. The storage distance for the left-turn movement group is computed using Equation 6.22. The storage distance for the right-turn movement group is computed using a variation of this equation.

$$N_{\text{qx,lt},n,k} = \frac{L_{a,\text{thru}} + L_{a,\text{lt}}(N_{\text{lt}} - 1)}{L^{*}_{h,k}} \tag{6.22}$$

where

$N_{\text{qx,lt},n,k}$ = maximum queue storage for left-turn movement group during subperiod k, veh;

$L_{a,\text{thru}}$ = available queue storage distance for the through movement, ft;

$L_{a,\text{lt}}$ = available queue storage distance for the left-turn movement, ft;

N_{lt} = number of lanes in the left-turn bay, lanes; and

$L^{*}_{h,k}$ = effective average vehicle spacing in stationary queue during subperiod k, ft/veh.

The available queue storage distance for the through movement equals the segment length less the width of the upstream intersection. For turn movements served from a turn bay, this length equals the length of the turn bay. For turn movements served from a lane equal in length to that of the segment, the queue storage length equals the segment length less the width of the upstream intersection.

The maximum queue storage for the through movement group is computed using Equation 6.23:

$$N_{\text{qx,lt},n,k} = \frac{L_{a,\text{thru}} + L_{a,\text{lt}}(N_{\text{lt}} - 1)}{L^{*}_{h,k}} \tag{6.23}$$

where $N_{\text{qx,thru},n,k}$ equals the maximum queue storage for through movement group during subperiod k, in vehicles, and N_{th} equals the number of through lanes (shared or exclusive), in lanes.

The initial queue for each movement group exiting a segment is compared with the maximum queue storage values. Any initial queue that exceeds the maximum value is set to equal the maximum value.

Step 6: Prepare for the Next Subperiod by Determining the Saturation Flow Rate Adjustment

During this step, the saturation flow rate is recomputed for movement groups entering the site identified in Step 3 as having spillback. This modification is necessary to prepare for a new evaluation of the facility during the next subperiod.

The process of recomputing the saturation flow rate uses an iterative loop. The loop converges when the saturation flow rate computed for each upstream movement is sufficiently small that the number of vehicles entering the spillback segment just equals the number that leaves it. To produce this result, a spillback saturation flow rate adjustment factor f_{sp} is computed for each movement. Its value is set to 1.0 at the start of the first loop (i.e., $f_{\text{sp},0} = 1.0$).

The process begins by setting the analysis time to equal the time remaining in the original analysis period (i.e., $T = T_o - T_{\text{total},k}$).

The next task is to compute the estimated volume arriving to each movement exiting the segment at the downstream signalized intersection (i.e., the adjusted destination volume). This calculation is based on the origin–destination matrix and discharge volume for each movement entering the segment. These quantities are obtained from the variables calculated using the HCM2010 methodology, as described in Section 1 in Chapter 30 of HCM2010. The adjusted destination volume is computed using Equation 6.24:

$$D_{a,j,k} = \sum_{i=1}^{4} v_{\text{od},i,j,k} \tag{6.24}$$

where

$D_{a,j,k}$ = adjusted volume for destination j ($j = 1, 2, 3, 4$) for subperiod k, veh/h; and

$v_{\text{od},i,j,k}$ = volume entering from origin i and exiting at destination j for subperiod k, veh/h.

The letters j and i in Equation 6.24 denote the following four movements: 1 = left turn, 2 = through, 3 = right turn, and 4 = combined mid-segment access points.

The next task is to compute the proportion of $D_{a,j,k}$ that originates from upstream origin i. These proportions are computed using Equation 6.25:

$$D_{a,j,k} = \sum_{i=1}^{4} v_{\text{od},i,j,k} \tag{6.25}$$

where $b_{i,j,k}$ is the proportion of volume at destination j that came from origin i for subperiod k, in veh/h.

The next task is to estimate the maximum discharge rate for each upstream movement. This estimate is based on consideration of the capacities of the downstream movements exiting the segment, and their volumes. When the segment has incurred spillback, the capacity of one or more of these exiting movements is inadequate relative to the discharge rates of the upstream movements entering the segment. The computed maximum discharge rate is intended to indicate the amount by which each upstream movement's discharge needs to be limited to maintain a balance between the number of vehicles entering and exiting the segment. Equation 6.26 is used for this purpose. They are applied to each of the four upstream entry movements i.

$$\mathrm{dv}_{u,i,k} = b_{i,2,k} \times c_{d,2,k} + \min(b_{i,1,k} \times c_{d,1,k}, \mathrm{fx}_{i,2,k} \times v_{\mathrm{od},i,1,k}) + \min(b_{i,3,k} \times c_{d,3,k}, \mathrm{fx}_{i,2,k} \times v_{\mathrm{od},i,3,k}) + \mathrm{fx}_{i,2,k} \times v_{\mathrm{od},i,4,k} \qquad (6.26)$$

with

$$\mathrm{fx}_{i,2,k} = \frac{b_{i,2,k} \times c_{d,2,k}}{v_{\mathrm{od},i,2,k}} \qquad (6.27)$$

where

$\mathrm{dv}_{u,i,k}$ = maximum discharge rate for upstream movement i for subperiod k, veh/h;
$c_{d,j,k}$ = capacity at the downstream intersection for movement j for subperiod k, veh/h; and
$\mathrm{fx}_{i,2,k}$ = volume adjustment factor for origin i for subperiod k.

The factor fx represents the ratio of two quantities. The numerator represents the downstream through capacity that is available to the upstream through movement. The denominator represents the volume entering the segment as a through movement and exiting as a through movement. The ratio is used to adjust the exiting turn movement and access point volumes such that they are reduced by the same proportion as is the volume for the exiting through movement.

The product $b_{i,j,k} \times c_{d,j,k}$ represents the maximum discharge rate for entry movement i that can be destined for exit movement j such that the origin–destination volume balance is maintained and the exit movement's capacity is not exceeded. It represents the allocation of a downstream movement's capacity to each of the upstream movements that use that capacity when the allocation is proportional to the upstream movement's volume contribution to the downstream movement volume.

The capacity for the combined set of access points is unknown and is unlikely to be the source of spillback. Hence, that capacity is not considered in Equation 6.26.

The next task is to estimate the saturation flow rate adjustment factor for the movements at the upstream signalized intersection. The movements of interest are those that enter the subject segment. Equation 6.28 is used for this purpose:

$$f_{\mathrm{sp},i,k,l} = \left(\frac{\mathrm{dv}_{u,i,k}}{c_{u,i,k}}\right)^{0.5} \times f_{\mathrm{ms},i,k} \times f_{\mathrm{sp},i,k,l-1} \qquad (6.28)$$

where

$f_{\mathrm{sp},i,k,l}$ = adjustment factor for spillback for upstream movement i for iteration l in subperiod k;
$c_{u,i,k}$ = capacity at the upstream intersection for movement i for subperiod k, veh/h; and
$f_{\mathrm{ms},i,k}$ = adjustment factor for downstream lane blockage for movement i for subperiod k.

The adjustment factor is shown to have a subscript l, indicating that the factor value is refined through an iterative process in which the factor computed in a previous iteration is updated using Equation 6.28.

In theory, the exponent associated with the ratio in parentheses should be 1.0. However, an exponent of 0.5 was found to provide for a smoother convergence to the correct factor value.

The adjustment factor for downstream lane blockage was described in an earlier section of this chapter. The discussion following Equation 6.14 noted that this adjustment factor is incorporated into the spillback factor (as shown in Equation 6.28) for segments with spillback.

The last task of this step is to adjust the access point entry volumes. Equation 6.29 is used for this purpose. One factor is computed for each access point movement that departs from the access point and enters the direction of travel with spillback.

$$f_{\mathrm{ap},m,n,i,k,p} = (\mathrm{fx}_{i,4,k})^{0.5} \times f_{\mathrm{ap},m,n,i,k,p-1} \qquad (6.29)$$

where $f_{\mathrm{ap},m,n,i,k,p}$ equals the access point volume adjustment factor for movement i at access point n of site m for iteration p in subperiod k.

The access point volume adjustment factors are used to adjust the volume entering the segment at each access point.

Step 7: Implement the HCM2010 Methodology to Evaluate the Remaining Time

The HCM2010 methodology is implemented in this step to evaluate the facility described by the input data. The analysis period was set in Step 6 to equal the time remaining in the original analysis period. The saturation flow rate of each movement influenced by spillback is adjusted using the factors quantified in Step 6.

Step 8: Compute the Queue Prediction Error

During this step, the predicted residual queue for each movement group is compared with the maximum queue storage.

This distance is computed using the equations described in Step 5. Any difference between the predicted and maximum queues is considered a prediction error. If the sum of the absolute errors for all movements is not equal to a small value, then the analysis returns to Step 6.

Step 9: Check the Total Time of Analysis

During this step, the total time of analysis $T_{\text{total},k}$ is compared with the original analysis period T_o. If they are equal, then the analysis continues with Step 10. If the two times are not in agreement, the access point volumes are restored to their original value and then multiplied by the most current access point volume adjustment factor. The analysis then returns to Step 2.

Step 10: Compute the Performance Measure Summary

During this step, the average value of each performance measure is computed. The value is a representation of the average condition for the analysis period. For uniform delay at one intersection, it is computed using Equation 6.30.

$$d_{1,i,j} = \frac{d_{1,\text{agg},i,j,\text{all}}}{T_o \times v_{i,j}} \tag{6.30}$$

where

$d_{1,i,j}$ = uniform delay for lane group j at intersection i, s/veh;
$d_{1,\text{agg},i,j,\text{all}}$ = aggregated uniform delay for lane group j at intersection i for all subperiods, s/veh; and
$v_{i,j}$ = demand flow rate for lane group j at intersection i, veh/h.

A variation of Equation 6.30 is used to compute the average value for the other intersection performance measures of interest.

Equation 6.31 is used to compute the average running time for one site, when a site is one direction of travel on one segment.

$$t_{R,m} = \frac{t_{R,\text{agg},m,\text{all}}}{\sum_{k=1}^{n} w_{\text{thru},m,k}} \tag{6.31}$$

where

$t_{R,m}$ = segment running time for site m, s;
$t_{R,\text{agg},m,\text{all}}$ = aggregated segment running time for site m for all n subperiods, s; and
$w_{\text{thru},m,k}$ = weighting factor for site m for subperiod k, veh.

A variation of Equation 6.31 is used to compute the average value for the other intersection performance measures of interest. The term in the denominator of Equation 6.31 equals the total through volume during the analysis period.

Procedure for Saving Performance Measures

The performance measures are computed using the HCM2010 methodology and saved at selected points in the spillback methodology. These measures typically correspond to a specific subperiod of the analysis period. Each measure is saved by accumulating its value for each subperiod. The sum is then used to compute an average performance measure value during the last step of the methodology.

Equation 6.32 is used to save the computed uniform delay for one intersection lane group. The computed delay represents a cumulative total time for the current, and all previous, subperiods.

$$d_{1,\text{agg},i,j,k} = d_{1,\text{agg},i,j,k-1} + d_{1,i,j,k} \times w_{i,j,k} \tag{6.32}$$

with

$$w_{i,j,k} = T \times v_{i,j,k} \tag{6.33}$$

where

$d_{1,\text{agg},i,j,k}$ = aggregated uniform delay for lane group j at intersection i for subperiods 0 to k, s/veh;
$d_{1,i,j,k}$ = uniform delay for lane group j at intersection i for subperiod k, s/veh;
$w_{i,j,k}$ = weighting factor for lane group j at intersection i for subperiod k, veh; and
$v_{i,j,k}$ = demand flow rate for lane group j at intersection i for subperiod k, veh/h.

The weighting factor represents the number of vehicles arriving during the analysis period for the specified lane group.

A variation of Equation 6.32 is also used to compute the aggregated values of the following performance measures at each intersection:

- Incremental delay;
- Initial queue delay;
- Uniform stop rate;
- Incremental stop rate based on second-term back-of-queue size; and
- Initial queue stop rate based on third-term back-of-queue size.

Equation 6.34 is used to save the computed running time for one site, when a site is one direction of travel on one segment.

$$t_{R,\text{agg},m,k} = t_{R,\text{agg},m,k-1} + t_{R,m,k} \times w_{\text{thru},m,k} \tag{6.34}$$

with

$$w_{\text{thru},m,k} = T \times \begin{pmatrix} v_{t,i,j,k}\, N_{t,i,j} + v_{\text{sl},i,j,k}\left(1 - P_{L,i,j,k}\right) \\ + v_{\text{sr},i,j,k}\left(1 - P_{R,i,j,k}\right) \end{pmatrix} \tag{6.35}$$

where

$t_{R,\text{agg},m,k}$ = aggregated segment running time for site m for subperiods 0 to k, s;

$t_{R,m,k}$ = segment running time for site m for subperiod k, s;

$w_{\text{thru},m,k}$ = weighting factor for site m for subperiod k, veh; and

$v_{t,i,j,k}$ = demand flow rate in exclusive through lane group j at intersection i for subperiod k, veh/h/lane;

$N_{t,i,j}$ = number of lanes in exclusive through lane group j at intersection i, lanes;

$v_{\text{sl},i,j,k}$ = demand flow rate in shared left-turn and through lane group j at intersection i for subperiod k, veh/h;

$v_{\text{sr},i,j,k}$ = demand flow rate in shared right-turn and through lane group j at intersection i for subperiod k, veh/h;

$P_{L,i,j,k}$ = proportion of left-turning vehicles in the shared lane group j at intersection i for subperiod k, and

$P_{R,i,j,k}$ = proportion of right-turning vehicles in the shared lane group j at intersection i for subperiod k.

When applying Equations 6.34 and 6.35, the lane group j and intersection i are located at the downstream end of the subject site m. The weighting factor represents the number of through vehicles arriving at the downstream intersection as a through movement during the analysis period.

A variation of Equation 6.34 is also used to compute the aggregated values of the following performance measures at each intersection:

- Through movement delay;
- Through movement stop rate;
- Travel time at free-flow speed; and
- Travel time at base free-flow speed.

The methodology described in this section is new to the urban streets methodology in Chapter 17 of the HCM2010. Incorporation of this methodology into the HCM requires its adoption by TRB's Highway Capacity and Quality of Service Committee.

The saturation flow rate adjustment factor for spillback is applicable to all signalized intersection movements that enter the urban street segment of interest. It is used to adjust the saturation flow rate of these movements. It would be added to Equation 18-5 in Chapter 18 of the HCM2010. Its implementation approach is the same as that described for the saturation flow rate adjustment factor for downstream lane blockage, as described earlier.

Validation of Urban Streets Model

This subsection describes the activities undertaken to validate the accuracy of the three enhancements described in this section. The objective of the validation process is to demonstrate the ability of the methodology to accurately predict urban street performance for a wide range of conditions. To facilitate the validation, the three enhancements were implemented in a test version of the computational engine that automates the urban streets methodology in Chapter 17 of the HCM2010.

The validation was based on a comparison of performance estimates from the engine with those obtained from a traffic simulation model. Three urban street segments were selected for the evaluation. The evaluation activities included the initial coding of a data file for each segment and the subsequent comparison of estimates from the engine and simulation model.

Table 6.9 describes the three segments. All three have coordinated-actuated control, and they offer a range in speed limit, access point density, median type, and segment length. All segments have four through lanes, two lanes in each direction. Table 6.10 summarizes the traffic characteristics and signalization at each of the segments.

The evaluation was based on a comparison of performance measures obtained from the enhanced HCM2010 methodology with those obtained from a simulation model. CORSIM (version 5.1) was determined to be a suitable simulation model for this purpose.

For each segment, spillback was created for one travel direction by reducing the number of through lanes by one. This direction is referred to as the *spillback direction.*

Through movement delay was used to assess the level of agreement between the simulated and predicted segment

Table 6.9. Segment Description

Segment	Location	Street Class	Segment Length (ft)	Speed Limit (mph)
Aviation Parkway	Tucson, Arizona	High-speed principal arterial	2,800	55
SW Barbur Boulevard	Portland, Oregon	Suburban principal arterial	2,937	35
SE Powell Boulevard	Portland, Oregon	Suburban minor arterial	1,405	35

Table 6.10. Segment Traffic and Signalization Characteristics

Segment	Travel Direction	Average Volume[a] (veh/h)	Base Free-Flow Speed[b] (mph)	Signal Timing[a] Cycle Length (s)	Signal Timing[a] Major-Street Split (s)	Left-Turn Phasing[a] Major Street	Left-Turn Phasing[a] Minor Street
Aviation Parkway	NB	1,200	47	80	54	Protected	Protected
	SB	1,300	47		54	Permissive	Permissive
SW Barbur Boulevard	NB	530	40	100	52	Protected	Split
	SB	1,700	40		54	Protected	Protected–Permissive
SE Powell Boulevard	EB	1,130	38	120	93	Protected	Permissive
	WB	1,147	38		99	Permissive	Permissive

Note: NB = northbound; SB = southbound; EB = eastbound; WB = westbound.
[a] Data apply to the downstream intersection.
[b] Base free-flow speed is computed using the procedure in Chapter 17 of the HCM2010 (TRB 2010a).

operation. Delay data were collected for the signalized intersection serving vehicles exiting the segment in the spillback direction. Thus, the delay to the four through movements at that signalized intersection was collected for each of three segments, yielding 12 observations. Turn movement delay was not collected because the HCM2010 methodology does not explicitly address the effect of turn bay blockage on turn movement delay.

The delay time estimate from CORSIM was compared with the HCM2010 control delay estimate. Delay time is computed as the difference between the total travel time and the travel time at the free-flow speed. This delay was determined to be the most appropriate delay variation for comparison with HCM2010 control delay for congested conditions.

The analysis period used with the HCM2010 methodology, and the total run time used with the simulation model, were both set at 1 hour. This approach was used because the delay incurred by oversaturated movements is time-dependent. Seven replications were used for the simulation of each segment. The average delay for the replications was compared with the results from the enhanced HCM2010 methodology.

The delay data obtained from the two sources are compared in Figure 6.18. The three data points shown in the "congested" region of the figure correspond to the through movement for the spillback direction at each of the three

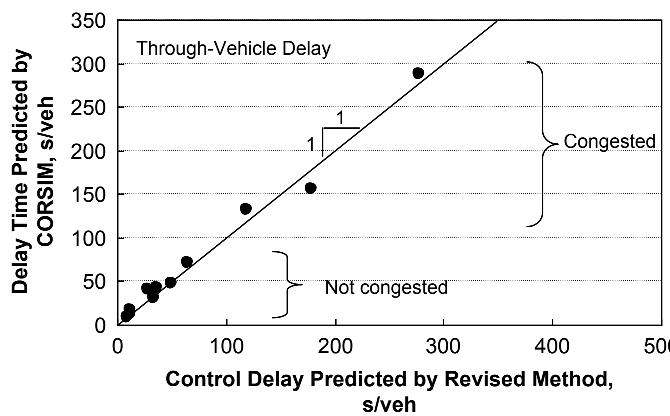


Figure 6.18. Control delay comparison for three segments.

segments. The other data points correspond to the through movements that are crossing the segment or traveling in the nonspillback direction.

The thin line shown in the figure represents an $x = y$ line. A data point that falls on this line indicates the engine value equals the simulation value. In general, the data tend to cluster around the $x = y$ line, suggesting that the engine prediction is in good agreement with that obtained from CORSIM.

CHAPTER 7

Corridor Applications

A corridor study, by definition, goes beyond the single-facility focus of a typical HCM2010 facility analysis. The purpose of a corridor study is to assess the ability of a subsystem of interrelated facilities to achieve a set of transportation performance objectives. The evaluation of travel time reliability at the corridor level introduces a new set of considerations that do not come into play when evaluating the reliability for individual facilities. These include

- The greater geographic coverage required to evaluate reliability at a corridor level, which imposes greater input data and data analysis requirements than for single-facility analyses;
- The interpretation of reliability results for different facility types within the same corridor; and
- The effect of varying congestion on facility demands (i.e., route shifting between facilities).

The latter consideration is the most likely reason for evaluating a corridor as a whole, rather than evaluating the individual facilities separately:

- Do some, all, or none of the facilities in the corridor provide at least minimum desired levels of reliability?
- Is localized congestion on the arterial that creates traffic diversion to the freeway a possible source of reliability problems on the freeway (or vice versa)?
- How well do the corridor's urban street components accommodate traffic diversion generated by incidents on the freeway?

This chapter provides guidance on extending the individual freeway facility and urban street facility reliability methods into an overall assessment of corridor reliability.

Corridor Definition

The HCM2010 defines a corridor as follows:

> Corridors are generally a set of parallel transportation facilities designed to move people between two locations. For example, a corridor may consist of a freeway facility and one or more parallel urban street facilities. There may also be rail or bus transit service on the freeway, the urban streets, or both, and transit service could be provided within a separate, parallel right-of-way. Pedestrian or bicycle facilities may also be present within the corridor, as designated portions of roadways and as exclusive, parallel facilities (TRB 2010a, 2-5).

For the purposes of a reliability analysis, a corridor is defined as a freeway facility and one or more parallel urban street facilities. When traffic diversion occurs between facilities that make up a corridor, the freeways, highways, and urban streets that cross the corridor and provide connections between the corridor's facilities are also affected; however, those effects are beyond the scope of a reliability analysis. The focus of a corridor evaluation is on the parallel facilities.

Methodological Considerations

The freeway facility and urban street facility reliability models use different methods to develop a travel time distribution for the reliability reporting period:

- The freeway facility method develops scenarios based on their probability of occurrence during the reliability reporting period. Some highly unlikely scenarios may be dropped from the analysis.
- The urban streets method randomly assigns demand, weather, and incident conditions to each day, based on distributions of conditions likely to occur within a month. Some

highly unlikely combinations may be included by random chance; therefore, multiple runs of the method may be needed to establish a representative travel time distribution.

Importantly, the two methods have no direct link. The weather pattern generated by the urban streets method may produce more or less severe conditions over a given model run, compared with the 10-year average weather conditions used by the freeway method. An incident scenario for the freeway does not generate a corresponding high-demand scenario for urban streets. When local data are used to generate demand patterns, traffic diversion effects appear in individual days' demands used to create month-of-day factors, but the effects of days with diversion are likely washed out by demands from all of the days without diversion. When default demand pattern data are used, no diversion effect occurs at all beyond that resulting from bad weather (and associated higher incident rates) which occurs more often in some months of the year than in others. Finally, the HCM2010 itself is incapable of predicting where and how much traffic diversion or change in demand (i.e., trips postponed or not made) will occur in response to incidents or severe weather events.

Given these differences, one could easily conclude that the freeway and urban streets reliability methods cannot be combined to describe corridor reliability. That would be incorrect. However, these limitations do have to be considered as one asks questions about corridor reliability and designs an analysis to answer those questions.

Potential Applications of Corridor Reliability Analysis

Evaluating Overall Corridor Reliability

An analysis of overall corridor reliability involves comparing selected reliability performance measures (e.g., TTI, PTI, percent on-time arrivals) generated for the individual facilities against either an established standard or comparative national values of reliability. Because different agencies may be responsible for different facilities within a corridor (or, in the case of urban streets, different portions of the facility), and because corridor analysis focuses on longer-distance travel, a regional standard may be most appropriate. In the absence of such a standard, a percentile threshold can be used so that unacceptable performance is defined in terms of, for example, a facility's PTI being among the worst 20% of U.S. facilities.

Once acceptable or unacceptable performance has been determined for each facility forming the corridor, the results can be interpreted as follows:

- All facilities have acceptable performance: The corridor offers acceptable reliability and provides travelers with multiple options for reliable travel.
- Some facilities have unacceptable performance: The corridor offers acceptable reliability and provides at least one option for reliable travel, although it may not be the fastest option on a typical day. These corridors may be candidates for variable message signing that provides travel times via both (or all) facilities to encourage the use of alternative facilities when one facility's travel time is considerably higher than the average.
- All facilities have unacceptable performance: The corridor's reliability is unacceptable—travelers cannot rely on any facility within the corridor to provide consistently reliable travel times.

Overall corridor reliability can be evaluated using actual travel time distributions to produce the reliability performance measures (thus accounting for traffic diversion effects directly) or by using this chapter's methods to generate travel time distributions (thus accounting for any traffic diversion effects manually, as described in the following text).

Prioritizing Corridor Management Strategies

This application focuses on measuring or estimating TTI, PTI, or both. These performance measures are compared with reliability performance measures to determine the percentile rank each facility in the corridor falls into (e.g., 25% worst freeway, 45% worst arterial). The rankings become inputs to a process for setting priorities (one that potentially incorporates other factors, such as vehicle miles traveled, or VMT). Worse-performing corridors are given higher priority to receive operational or physical treatments designed to improve travel time reliability.

Diagnosing Potential Recurring Diversion-Related Causes of Unreliability

In this application, each facility forming the corridor is divided into sections defined by the locations of connections between facilities (i.e., locations where a freeway and an urban street facility cross, or where another facility provides a connection between a freeway and an urban street). For freeways, sections end immediately before the basic freeway segment in the middle of the interchange. For urban streets, sections end at the downstream end of the segment that includes either (1) the cross-street intersection or (2) the intersection serving the freeway off-ramp in the analysis direction, in cases where the freeway and parallel urban street cross each other, as shown in Figure 7.1.

Next, the TTI is determined for each facility on a section-by-section basis. In some cases, on parallel sections of freeway and urban street, the TTI drops significantly from one section to the next on both facilities. Those sections are candidates for investigation to see whether operational problems on one

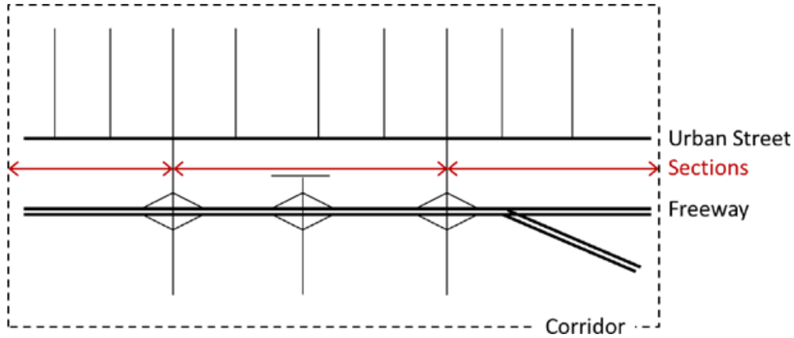


Figure 7.1. Illustration of a corridor.

facility cause regular traffic diversion to the other facility, resulting in problems on the other facility. Treating the source cause of unreliability can result in reliability improvements on both facilities.

Identifying the Potential for Nonrecurring Traffic Diversion

Parallel arterials are potential alternate routes for freeway traffic when incidents occur on the freeway. However, motorists may know they are better off sitting in traffic on the freeway because taking the urban street on an average day is slower than the freeway on its 5% worst day. If motorists know this, then little diversion is likely to occur—even if they are encouraged through information strategies such as variable message signs or highway advisory radio.

In this application, the facilities forming the corridor are divided into sections as before. This time, a planning travel time to the corridor end is determined for each freeway section based on the 95th percentile time to travel from the start of the freeway section to the end of the corridor. An average travel time to the corridor end is determined for each urban street section based on the 50th percentile travel time from section start to corridor end. Or, if data are available, the average time to travel from the freeway to the urban street along the roadway connecting the two facilities can be calculated and incorporated into the urban street section's time to corridor end. Sections in which the arterial street travel time to corridor end is greater than the freeway planning time to corridor end would not be candidates for operations measures that encourage traffic diversion.

Modeling Freeway Traffic Diversion Effects on Urban Streets

As noted previously, the freeway and urban streets methods are not linked: an incident in a freeway scenario does not generate a corresponding demand increase in an urban street scenario. However, for the purposes of modeling corridor operations, freeway incidents that would cause diversion can be modeled as special events in the urban streets model. This allows the urban streets reliability results to reflect demand increases resulting from freeway incidents.

Two key aspects of this method are notable. First, the analyst is responsible for estimating the amount of diversion that would occur—HCM2010 methods do not estimate demand or demand diversion. Second, the process has no feedback loop that allows the diverted demand to be subtracted from the freeway demand so that freeway reliability can be reestimated.

The following steps describe a conceptual process for determining freeway traffic diversion effects on urban streets reliability.

Step 1: Identify Number of Freeway Incidents During Reliability Reporting Period

The scenario generation step of the freeway modeling process is performed as described previously in this report. The process determines probabilities of an incident occurrence by month and incident severity and specifies the average duration. The average number of freeway incidents of severity can be determined from Equation 7.1:

$$I_s = \frac{h_{\text{RRP}} \times P(I_s)}{d_s} \tag{7.1}$$

where

I_s = number of freeway incidents of severity s generated during the reliability reporting period;
h_{RRP} = number of hours in the reliability reporting period;
$P(I_s)$ = probability of an incident of severity s; and
d_s = average duration of an incident of severity s.

Because a large number of shoulder and one-lane-closed incidents can be generated, and because more-severe incidents are most likely to generate traffic diversion, analysts should focus on incidents affecting two or more lanes.

Step 2: Apply the Freeway Reliability Method

Next, the freeway facility reliability method is applied. The scenario characteristics for scenarios containing incidents of interest to the analysis (e.g., two lanes closed and more severe) and the computational engine output for those scenarios should be saved for use in subsequent steps.

Step 3: Generate Incident Data

Incident Day and Time

Because the freeway model develops probabilities of incident occurrence, while the urban streets model generates conditions day-by-day during the reliability reporting period, each freeway incident needs to be assigned to a specific day and time within the reliability reporting period. This can be done randomly. As the freeway model defines two possible incident start times—at the start of the study period or in the middle of the study period—one of these times should be assigned randomly.

Incident Location

The freeway model assigns locations as the first, middle, or last segment of the freeway facility, with equal probability. The choice of freeway segment should be selected randomly. All diverted freeway demand is assumed to leave the freeway at the last cross-connection point (i.e., section boundary) before the incident and to return at the first opportunity (i.e., at the start of the following section). The corresponding arterial section is the one affected by the diverting traffic.

Corresponding Freeway Scenarios

The analyst needs to locate the input data for the scenario corresponding to the incident month, nonsevere weather, incident location, incident duration, incident start time, and average incident duration. Freeway demand and capacity data for the time the incident is in effect are required in the next step.

Diversion Duration

If the incident results in freeway traffic demand exceeding the remaining capacity, diversion occurs. Traffic starts to divert in the first analysis period after the incident occurs.

Amount of Diverted Traffic

The analyst needs to specify the amount of traffic by analysis period that diverts as a result of a given incident. This amount is likely proportional to the amount to which freeway demand exceeds capacity. It also includes assumptions about the methods used to communicate information about the incident to motorists.

Step 4: Code Alternative Data Sets

An alternative data set needs to be developed for each incident. The data set should specify changes in turning demand at the intersections that form urban street section boundaries, along with changes in through demand for segments and intersections within the urban street section. If special signal timing plans are used on the urban street during diversion situations, these should also be entered into the alternative data set.

Step 5: Apply the Urban Streets Reliability Method

Next, the urban streets reliability method is applied. Each alternative data set is supplied to the procedure as a special event, with its schedule corresponding to the selected day, start time, and duration of each diversion event.

The statistics produced by the urban streets method now include the effects of diverted freeway traffic in addition to all the other sources of variability normally accounted for by the method.

CHAPTER 8

Recommendations

This chapter recommends the next steps to carry forward the results of this research. The following topics are addressed:

- Defining reliability levels of service;
- Implementing the L08 research;
- Identifying freeway facility research needs; and
- Identifying urban streets research needs.

Defining Reliability Levels of Service

Reliability can be used to define level of service (LOS) in a variety of ways. The intent of this section is to identify a preferred approach for defining LOS based on the concept of reliability. LOS definitions require that cutoff points (boundaries) of the measurement unit be established that define each LOS range. The research team established the ranges described in this section to be illustrative, not definitive. The final selection of ranges will require additional analysis and, more important, a vote and approval by the Highway Capacity and Quality of Service Committee. The research team also proposed one option for defining reliability LOS (with input from the project panel), but the decision to adopt a formal definition lies with the Highway Capacity and Quality of Service Committee.

All reliability measures should be consistently scoped for accurate comparisons between facilities. Reliability measures created from observations during peak periods or peak hours vary greatly from measures created using 24-hour observations. The temporal extent of the analysis relative to the peak period(s) therefore critically affects the values of reliability measures. Facility selection (i.e., the definition of start and end points of the facility relative to the critical bottleneck) also plays a part in the reliability measures.

Options for Reliability LOS

Options for defining reliability-based LOS fall into four categories:

- Freeway reliability based on current LOS ranges;
- Freeway and urban streets reliability LOS based on travel speed ranges;
- Freeway reliability LOS based on most-restrictive condition; and
- Freeway and urban streets reliability LOS based on the value of travel.

Each option is discussed in more detail in the following subsections and illustrated using example data from various U.S. facilities.

Option 1: Freeway Reliability LOS Based on Current LOS Ranges

The simplest method for defining reliability LOS is to use the existing LOS definitions for freeway facilities and basic freeway segments—based on density—and urban streets and urban street segments—based on travel speeds.

For each facility type, the analysis procedure produces a distribution of the LOS measure that represents the percentage of trips (or the percentage of analysis periods) that fall into each LOS range. Alternately, the definition can be based solely on the percentage of trips (or percentage of analysis periods) in LOS F alone. Table 8.1 shows examples of this approach using freeway detector data from Seattle, Washington, and Atlanta, Georgia. Both the peak (northbound direction, in both cases) and off-peak directions are shown. For example, if reliability is based solely on the percentage of trips operating under LOS F, then the

Table 8.1. Density-Based Reliability LOS Based on Free-Flow Speed

LOS	Facility Density (pcpmpl)	Percentage of Trips in Each LOS Range, Weekdays, 4:30–6:00 p.m.			
		Seattle, I-405 (2007)		Atlanta, I-75, Northside (2010)	
		NB	SB	NB	SB
A	≤11	0.0	0.0	0.9	1.1
B	>11–18	8.1	1.4	0.4	1.9
C	>18–26	91.6	32.9	1.6	5.8
D	>26–35	0.3	34.0	2.7	69.8
E	>35–45	0.0	20.6	4.7	18.4
F	>45 or d/c > 1.0	0.0	11.1	90.7	3.0
Mean TTI		**1.016**	**1.352**	**1.984**	**1.050**

Note: pcpmpl = passenger cars per mile per lane; NB = northbound; SB = southbound; d/c = demand-to-capacity ratio.

southbound approach for the Seattle facility is "unreliable" 11.1% of the time (i.e., its reliability is 88.9%). Similar distributions can be constructed for urban streets and signalized intersections using the current LOS ranges in the HCM2010.

Density-based LOS for freeways is a significant departure from the concept of travel time reliability. The primary issue is that travel times do not vary much over a wide range of density-based LOS ranges. Further, by using the current freeway LOS ranges, the focus is on unsaturated (uncongested) conditions: LOS A through E is in the unsaturated range while all oversaturated conditions are grouped into a single LOS F category. Finally, the lower-density thresholds for weaving sections further complicate the use of density as the fundamental measure of reliability.

For urban streets, the definition would be consistent with the measurement of travel times (travel speeds over a distance of highway are derived from travel times). Travel times are not relevant for signalized intersections since they are "points" on the highway system; but for highway segments, delay is related to travel times as well.

Option 2: Freeway and Urban Streets Reliability LOS Based on Travel Speed Ranges

In this approach, travel speed ranges can be constructed for freeways in a manner similar to that for urban streets. Here, travel speed is analogous to space mean speed (SMS) over the entire freeway facility or segment. The LOS ranges may be based on percentages of the free-flow speed as they are for urban streets, or they may be set at fixed SMS values. Table 8.2 shows the values used for urban streets.

Table 8.2. LOS Ranges for Urban Street Facilities and Segments

Travel Speed as a Percentage of Free-Flow Speed (mph)	LOS by Critical v/c Ratio	
	≤1.0	>1.0
>85	A	F
>67–85	B	F
>50–67	C	F
>40–50	D	F
>30–40	E	F
≤30	F	F

Note: v/c = volume to capacity.
Source: HCM2010, Table 16-4 (TRB 2010a).

Because of the insensitivity of travel speeds to a wide range of density and volume-to-capacity (v/c) values (current LOS A through D), one option is to extend the number of LOS ranges for oversaturated conditions. An example of how this method would be applied is shown in Table 8.3, again using detector data from Seattle and Atlanta. Under this option, the southbound direction for the Seattle facility would be at LOS F (or "unreliable") 17.8% of the time.

Table 8.3. Freeway Reliability LOS Defined by Travel Speed or Travel Time Index Ranges

LOS	Travel Speed (mph)[a] and Equivalent TTI	Percentage of Trips in Each LOS Range, Weekdays, 4:30–6:00 p.m.			
		Seattle, I-405		Atlanta, I-75, Northside	
		NB	SB	NB	SB
A	≥60 (TTI ≤ 1.083)	27.7	1.6	0.4	15.4
B	50–59 (1.083 < TTI ≤ 1.300)	71.9	48.3	6.6	80.5
C	45–49 (1.300 < TTI ≤ 1.444)	0.3	12.0	3.2	1.4
D	40–44 (1.444 < TTI ≤ 1.625)	0.0	9.3	8.5	0.3
E	35–39 (1.625 < TTI ≤ 1.857)	0.1	11.0	14.4	0.8
F	<35 (TTI > 1.857)	0.0	17.8	66.8	1.6
Mean TTI		**1.016**	**1.352**	**1.984**	**1.050**

Note: NB = northbound; SB = southbound.
[a] Average speed over the length of the facility (i.e., the space mean speed).

Option 3: Freeway Reliability LOS Based on Most-Restrictive Condition

Options 1 and 2 are predicated on providing a distribution of the percentage of time the facility is unreliable, rather than assigning a single "grade" to define a highway's LOS. Although the distribution is highly useful for analysts, it may confuse nontechnical audiences who are used to using a single LOS value. Focusing on the percentage of trips/analysis periods in LOS F, rather than specifying the percentage in each LOS range, is a departure from how LOS ranges are defined in the HCM2010.

An alternative is to report the most restrictive condition. In this approach, travel speed boundaries are again defined, but the observation value is the percentage of trips greater than or equal to each travel speed. An example is shown in Table 8.4.

This approach requires setting a second threshold value: the cumulative percentage of trips for the restrictive condition. The analyst reads down the table, starting from LOS A and finds the LOS at which the cell value is greater than or equal to the threshold. For example, if the analyst wants 75% of trips to be in the most restrictive range, the data in Table 8.4 yields the following results:

- Seattle I-405 NB: LOS B;
- Seattle I-405 SB: LOS E;
- Atlanta I-75 NB: LOS F; and
- Atlanta I-75 SB: LOS B.

This option is functionally equivalent to selecting a percentile value for a threshold and seeing where it falls in the table. In Table 8.4, using the value at which 75% of trips fall is equivalent to calculating the 75th percentile TTI, converting it to travel speed (i.e., space mean speed), and finding the range in which it falls. Alternately, instead of using the 75th percentile TTI, the 25th percentile SMS can be used. Table 8.5 shows the data for this method.

A variant on this approach is to use a reliability metric for the threshold values. This option also establishes reliability LOS on the basis of a single value but in a simpler manner. It establishes LOS ranges for a reliability metric and makes the assignment solely on the basis of where the facility's calculated value falls. For illustration, the following example uses the planning time index (PTI) as the reliability metric; this is the ratio of the 95th percentile travel time to the free-flow or "ideal" travel time. Given the ranges shown in Table 8.6, the reliability LOS for the Seattle and Atlanta analysis sections can be assigned. For example, the LOS for the southbound direction of the Seattle facility is F.

Table 8.5. Percentile Values Matched to Speed Ranges

Section	75th Percentile TTI	Corresponding Travel Speed[a] (mph)	LOS (from Table 8.4)
Seattle, I-405 NB	1.018	58.9	B
Seattle, I-405 SB	1.545	38.8	E
Atlanta, I-75 NB	2.338	25.7	F
Atlanta, I-75 SB	1.037	57.9	B

Note: NB = northbound; SB = southbound.
[a] Based on free-flow speed of 60 mph.

Table 8.4. Freeway Reliability LOS Based on Most-Restrictive Condition

LOS	Travel Speed (mph)	Cumulative Percentage of Trips in Each Range, Weekdays, 4:30–6:00 p.m.			
		Seattle, I-405		Atlanta, I-75, Northside	
		NB	SB	NB	SB
A	≥60	27.8	1.6	0.4	15.4
B	≥50	99.6	49.9	7.0	95.9
C	≥45	99.9	61.8	10.3	97.3
D	≥40	100.0	71.2	18.8	97.7
E	≥35	100.0	82.2	33.2	98.4
F	≥30	100.0	92.2	51.2	99.5
Mean TTI		**1.016**	**1.352**	**1.984**	**1.050**

Table 8.6. Freeway Reliability LOS Based on the Planning Time Index

LOS	PTI	Calculated PTI, Weekdays, 4:30–6:00 p.m.			
		Seattle, I-405		Atlanta, I-75, Northside	
		NB	SB	NB	SB
A	1.00–1.10	1.032			
B	>1.10–1.25				1.155
C	>1.25–1.50				
D	>1.50–1.75				
E	>1.75–2.00				
F	>2.00		2.129	3.035	
Mean TTI		**1.016**	**1.352**	**1.984**	**1.050**

Note: NB = northbound; SB = southbound.

Generally speaking, all SMS measures can be converted to TTI for consistency. LOS can be defined on the basis of the full distribution of TTI; on the fraction of time TTI exceeds a given value (associated with LOS F); or on the basis of a range at a specified TTI percentile—for example, the 75th, 80th, or 95th percentile.

Option 4: Freeway and Urban Streets Reliability LOS Based on the Value of Travel

A somewhat radical departure from traditional LOS philosophy, which is based on performance from the perspective of the facility, is to base LOS on the value of travel perceived by users. The concept is to translate the values of both typical (average) travel time and travel time reliability into travel time equivalent values and then assign a cost to them. The LOS ranges are then based on unit costs per traveler. This approach can be applied to both interrupted and uninterrupted facilities.

The valuation approach is based on the work of Small et al. (2005). They define the reliability ratio as the value of reliability (VOR) divided by the value of time (VOT). SHRP 2 Project C04 suggests a range of 0.5 to 1.5, but a review of past studies suggests that it is more in the 0.9 to 1.2 range. Therefore, a value of 1.0 seems reasonable for composite trips. However, previous research also indicates that the value of reliability varies by trip purpose. Because the research to date has been limited, the value of the reliability ratio is still uncertain. Small et al. (2005) adopted the quantitative measure of variability as the upper tail of the distribution of travel times, specifically, the difference between the 80th and 50th percentile travel times. They argue that this measure is better than a symmetric standard deviation since, in most situations, being late is more crucial than being early, and many regular travelers build a safety margin into their departure time to leave them an acceptably small chance of arriving late (i.e., planning for the 80th percentile travel time means arriving late only 20% of the time). On this basis, *travel time equivalents* can be defined and used to put both typical (average) and reliability components into the same units. That is, reliability is equilibrated to average travel time.

The calculation of travel time equivalents is shown in Equation 8.1.

$$\mathrm{TT}_e = \mathrm{TT}_m + a \times (\mathrm{TT}_{80} - \mathrm{TT}_{50}) \quad (8.1)$$

where

TT_e = travel time equivalent on the segment or facility;

a = reliability ratio (VOR/VOT), set equal to 1.0 for now;

TT_m = mean travel time;

TT_{50} = 50th percentile travel time; and

TT_{80} = 80th percentile travel time.

Table 8.7 shows the results of applying this procedure. The end result is an estimate of an equivalent delay value, normalized to segment length (delay per mile). The LOS ranges can then be set on delay per mile.

This approach has the advantage of creating a single composite value for facility performance. In addition to deviating from traditional HCM LOS philosophy, the nascent nature of reliability valuation research is a problem in that future work is likely to produce different calculation methods and reliability ratios.

Summary of Options

Option 1: Reliability LOS Based on Current LOS Ranges

This option is the most consistent with current LOS concepts in the HCM2010. For urban streets and urban street segments, the current LOS ranges based on travel speeds can be used to present an LOS distribution (percentage of trips in each LOS range).

For freeways, basing reliability LOS on the current density-based LOS designations is not useful. Travel times are most variable under congested conditions, and the current density ranges for LOS A through E do not result in much change in travel times.

Table 8.7. Travel Time Equivalents and Equivalent Delays for Use in Setting Reliability LOS

	Travel Time (min)						
Section	Mean	80th Percentile	50th Percentile	TT_e	Free-Flow Travel Time	Excess Travel Time (delay)	Delay per Mile
I-405, NB	5.95	5.98	5.94	5.99	5.77	0.22	0.038
I-405, SB	6.67	8.13	5.91	8.89	4.94	3.95	0.800
I-75, NB	10.90	13.55	10.53	13.92	5.51	8.41	1.526
I-75, SB	6.13	6.07	5.97	6.23	5.84	0.39	0.067

Note: NB = northbound; SB = southbound.

However, creating a distribution rather than a single LOS value can be difficult to communicate to nontechnical audiences (a major use of the LOS concept). A simple solution is to report only the percentage of trips in LOS F or E + F, but this misses the remainder of the LOS distribution. While a single value is clearly more consistent with HCM2010 practices, the use of a distribution appears to lend itself to a reliability analysis. A reliability analysis inherently captures a range of operating conditions on the same facility and attributes those conditions to various sources of (un)reliability. Using a distribution of LOS values therefore intrinsically mirrors the variability of traffic conditions on the facility.

Option 2: Freeway Reliability LOS Based on Travel Speed Ranges

This option would make freeway reliability LOS conceptually consistent with urban streets and urban street segments. The problem of presenting a distribution rather than a single LOS value is still present.

Option 3: Freeway Reliability LOS Based on Most-Restrictive Condition

This method avoids the problem of presenting a distribution and assigns a single LOS value. It is more complicated to apply and explain in that two values must be set: a percentage threshold for the trips that fail to meet LOS criteria and the ranges for each LOS category.

Option 4: Reliability LOS Based on the Value of Travel

This option is the most complicated to both develop and explain. It has the advantage of being based on travelers' perception of reliability, but it relies on a factor (the reliability ratio) that has not been precisely identified and will likely change with new research. In addition to its complexity, establishing LOS ranges on the basis of travel time equivalents is highly problematic.

Recommended Option

Testing the four options with field data did not reveal a clearly better choice on which to base reliability LOS. Further, the research team found the four options to be difficult to communicate to the profession, the public, and decision makers. As a result, the team decided to develop an "on-time" measure similar to Option 2. This measure, the reliability rating, is the percentage of trips served at or below a threshold TTI (the ratio of actual travel time to free-flow travel time). The selected thresholds are 1.33 for freeways and 2.50 for urban streets. These thresholds approximate the points at which most travelers would consider a facility congested; thus, the measure roughly reflects the percentage of trips on a facility that experience conditions better than LOS F. The difference in threshold TTI values results from differences in how free-flow speed is defined for freeways compared with urban streets, as TTI is measured relative to free-flow speed.

The research team has not defined a service measure for travel time reliability. Because travel time reliability is a new concept for the transportation profession, the research team recommends that performance measures be used to describe the travel time reliability performance on freeways and urban streets. Subsequently, consideration can be given to using travel time reliability to define LOS. When reliability is considered as a service measure, the team recommends that the reliability rating (now a performance measure) be the basis.

Other considerations for future reliability LOS deliberations follow.

Urban Streets

Figure 16-4 of HCM2010 defines LOS F as either (1) where travel speed is 30% or less of the base free-flow speed or (2) where the subject through movement at one or more intersections has a v/c ratio greater than 1.0. Because the LOS definition is based on travel speed, which is a derivative of travel time, no changes in the LOS concept for urban streets is needed.

Freeways

For freeway reliability, the research team first recommends that the existing density-based LOS definition be replaced with a travel speed–based definition. Density should be maintained as the indicator of general freeway performance, especially for rural facilities. The team also recommends that, at some point in the future, travel speed be considered as a replacement for density even for general performance on urban facilities. The use of travel speed as the indicator of both general and reliability performance on freeways also provides consistency with the urban streets method.

Implementing the L08 Research

The draft HCM reliability chapters and computational engines (FREEVAL-RL and STREETVAL) were completed in draft form in the fall of 2012. The materials were fully vetted and reviewed by the TRB Highway Capacity and Quality of Service Committee in conjunction with the 2013 TRB annual meeting.

The computational engines consist of spreadsheets with embedded Visual Basic code. Separate Excel spreadsheet tools are used to generate the scenarios and run the FREEVAL and

STREETVAL engines, thus executing the HCM2010 calculations in an automated fashion and processing the results for reliability reporting purposes. Although not part of the L08 project, a natural extension of the computational engines and other tools would be the development of a more user-friendly, integrated software tool that could execute the files faster than the Excel-based computational engines. Such a software tool could be hosted on a fast server and located in any secure environment, including a cloud-based environment. Currently, the updated FREEVAL and new STREETVAL computational engines are hosted in the developer's environment at the contractor's site. No decision has been made on the hosting arrangement for the final product.

Identifying Freeway Facility Research Needs

Research needs in the freeway facilities methodology take into consideration improvements to the core HCM2010 methodology and to the reliability submodels developed in the course of this study.

Research to Overcome Core Methodology Limitations

While the freeway facility methodology has been significantly improved and expanded in the course of this study, it still needs additional research to fill some significant gaps.

Oversaturated Model

The oversaturated flow-density relationship has not been calibrated since its inception in the 2000 HCM. Several research efforts have compared the results of the HCM2010 method with field observations in an effort to validate the predicted performance. Nonetheless, a more rigorous calibration effort is desired, with the potential of enhancing the current linear speed–flow relationship used to model operations at densities greater than 45 pcphpl.

Off-Ramp Spillback Modeling

Spillback from off-ramps is not considered in the current methodology, significantly weakening its ability to model congested corridors. Off-ramps are often choke points along freeway facilities, especially in the case of a freeway–arterial corridor pair, as a signalized intersection may cause spillback onto the freeway. The current implementation uses only a simple off-ramp capacity check, without further scrutiny of the impacts on the freeway. An enhanced off-ramp spillback model should be sensitive to queuing patterns on the freeway, which are likely to use only some of the freeway lanes (depending on the cross-section), and should also consider speed drops in the adjacent nonqueued lanes.

Free-Flow Speed and Capacity Effects

The free-flow speed and capacity adjustment factors used throughout the methodology to account for nonrecurring congestion effects have been adopted from the most recent and relevant literature, but they have not been locally calibrated or validated. While a literature synthesis is an appropriate approach for a project like this, it carries the risk of inconsistencies in parameter definitions and data collection. A coordinated research effort would allow for a consistent evaluation of these effects and should carry a special emphasis on interaction effects, such as inclement weather in work zones, or incidents on inclement weather days.

Managed Lane Modeling

While the methodology developed in this project does not explicitly incorporate the new method for analyzing managed lanes completed under NCHRP Project 3-96, analysts may use the 3-96 results to calibrate this methodology's base facility inputs. Specifically, the 3-96 method introduces the concept of two parallel lane groups on a freeway facility, distinguishing between general purpose and managed lanes. The 3-96 method generally does not change the underlying methodologies for general purpose lanes; rather, it emphasizes the development of speed–flow curves and friction effects for managed lanes. The methodology is modeled so that managed lane operations affect general purpose lanes in cases of "cross-weave friction" resulting from at-grade access points to and from the managed lanes.

Analysts wishing to perform a reliability analysis on the general purpose portion of a managed lane facility should calibrate the base facility performance by using the 3-96 method and, to implement additional capacity adjustment factors in the L08 method, should the presence of access points result in significant friction impacts on the general purpose lanes. The analyst would run the base facility seed file in both the FREEVAL-RL and the FREEVAL-ML engines, and then calibrate the performance of FREEVAL-RL to match the FREEVAL-ML friction effects.

Research to Improve the Reliability Submodels

Demand Impacts of Nonrecurring Congestion

Research is needed to understand and quantify the effects of weather, work zones, and special events on traffic demand. The current demand variability is a function of the day of the week and the month of the year; as such, it accounts for some implicit correlation between, for example, weather and demand (e.g., more snow in the winter and generally lower traffic demands). However, no explicit modeling has accounted for the impacts of weather, work zones, and special events, although they are intuitively expected to reduce demand through diverted trips, carpooling, and other effects. Similarly, work zones are expected to affect facility demand. Depending on the level of penetration of traveler information systems, even incidents may result in demand shifts. Intuitively, all of these effects have an impact on the reliability of the facility and should be considered in future research. Ignoring those sensitivities may result in overestimating the impact of nonrecurring congestion on reliability performance measures.

Conditional Probabilities of Submodels

The L08 method assumes that incident rates and weather conditions are independent. The method does account for the possibility of incidents during inclement weather events but assumes a simple multiplication of the underlying probabilities. Research is needed to develop models that explain the relationship and to derive conditional probabilities of incidents under different weather conditions, as well as incidents in work zones.

Enhanced Weather Detail

The methodology does not currently account for weather events that have a small effect on segment capacity reduction (<2%). In addition, a given weather event (e.g., rain, snow) is always assumed to occur at its mean duration value; and only two possible start times for weather events are considered. Although the low-capacity impact scenarios are incorporated in the general "nonsevere" weather category, the methodology would benefit from added detail on weather duration and weather starting times—both of which can be explored in future research.

Enhanced Incident Detail

To consider the average effect of incidents on a facility, the analyst assumes each incident is located on one of three possible segments: the first segment, the segment at the facility midpoint, or the last segment. Similarly, the timing of each incident is set as either the start of a study period or its midpoint. Finally, only three possible incident durations are considered: the 25th, 50th, and 75th percentiles of the incident duration distribution. These assumptions on incident location, starting time, and duration were essential in enabling the team to enumerate a discrete (and manageable) number of reliability scenarios. Future research may explore options for stochastic incident modeling in a reliability context, using segment-specific incident probabilities.

Identifying Urban Streets Research Needs

The research conducted for this project has led to the formulation of several recommendations for future research on urban streets methodology. That research is grouped into two categories. The first category describes the research needed to overcome known limitations in the scope of the urban streets reliability methodology. The second category describes research needed to improve specific models within the reliability methodology.

Research to Overcome Limitations

In general, the urban streets reliability methodology can be used to evaluate the performance of most urban street facilities. However, the methodology does not address some events or conditions that occur on some streets and influence their operation. These events and conditions are identified in the following paragraphs.

Calculation of Facilitywide Performance Measures

The HCM2010 urban streets methodology predicts the travel time and speed of *through* vehicles (i.e., vehicles traveling along the facility and served as a through movement at each intersection). However, it does not describe a procedure for aggregating the performance of all movements on each segment and that of all movements on each external intersection approach. This type of estimate would describe facilitywide performance and include measures such as vehicle miles traveled (VMT), vehicle hours traveled (VHT), and vehicle hours delay (VHD), and their equivalents for person movement (PMT, PHT, and PHD).

Facilitywide performance measures are critical for investment and planning studies that compare alternatives and select the most cost-effective alternative. These measures are also critical building blocks for a future HCM corridor methodology. The use of PMT, PHT, and PHD would facilitate assessments of the service provided to transit passengers, bicycle riders, and pedestrians, as well as auto drivers and passengers.

The extended HCM2010 urban streets method should address speed estimates for nonthrough vehicles on the segment (which are not currently covered in the HCM2010). In addition, the extended method should quantify the impacts of vehicles denied entry to the facility during each analysis period because of severe congestion within the facility. These effects could be quantified in terms of total VHD and PHD during the entire study period.

Truck Pick-Up and Delivery

Lane and shoulder blockages resulting from truck pick-up and delivery activities in downtown urban areas can be considered like incidents in terms of the randomness of their occurrence and duration. The dwell time for these activities can range from 10 min to 20 min. They are estimated to result in 950,000 VHD on the nation's urban arterial streets (Chin et al. 2004).

Research is needed to quantify the effect of truck pick-up and delivery activities on the speed and capacity of an urban street segment. The research should also develop models for predicting the frequency and duration of such activities. The scope of the research may be broadened to include the effect of on-street parking on urban street operation. The research results should be suitable for reliability evaluation.

Signal Malfunction

A signal malfunction occurs when one or more elements of the signal system are not operating in the intended manner. These elements include vehicle detectors, signal heads, and controller hardware. A failure of one or more of these elements typically results in poor facility operation. For example, a detector failure typically causes a fail-safe operation in which a continuous call is held by the detection system, thereby extending the subject phase to its maximum green limit. A failure to a signal head or the controller hardware can result in a flashing-red operation, making traffic control equivalent to all-way-stop control. A failure to the communications system can lead to loss of signal coordination. Anecdotal information indicates that between 10% and 20% of an agency's detection sensors are not functioning at any particular time.

Research is needed to quantify the effect of signal malfunction on the operation of an urban street segment. The research should focus on the more common types of malfunction. It should separately quantify the effect of each type on speed, saturation flow rate, and other traffic characteristics that influence urban street operation. The research should also develop models for predicting the frequency and duration of common types of malfunction. The research results should be suitable for reliability evaluation.

Railroad Crossing and Preemption

Chin et al. (2004) used data from the Federal Railroad Administration to estimate the nationwide delay related to railroad crossings. Their evaluation considered all crossings on urban principal arterials, regardless of whether they occurred at a signalized intersection. They estimated the delay to be 2,700,000 vehicle hours. This amount is relatively small in the context of other sources of urban street congestion (e.g., incidents, weather, work zones). Nevertheless, train crossing times can be lengthy (typically 5 min to 10 min) and can result in considerable delay at each crossing.

A railroad crossing at a mid-segment location on an urban street facility effectively blocks traffic flow while the train is present. Urban street operation can also be disrupted when a train crosses a cross-street leg of a signalized intersection. Signal coordination may be disrupted for several cycles following train clearance.

Research is needed to quantify the effect of railroad crossings on urban street operation. The research should address both mid-segment crossings and intersection preemption resulting from a cross-street crossing. The research should develop a procedure for quantifying the effect of train events on speed, saturation flow rate, and other traffic characteristics that influence urban street operation. The research should also develop models for predicting the frequency and duration of train crossings and preemption events. The research results should be suitable for reliability evaluation.

Adverse Weather Conditions

The current methodology does not address weather conditions that restrict driver visibility or degrade vehicle stability. These conditions include fog, dust storms, smoke, and high winds. Chapter 10 of the HCM2010 indicates that a significant visibility restriction can reduce freeway capacity by about 10%. It also indicates that high winds can reduce freeway capacity by 1% to 2%. The impacts of these conditions on urban street operation are unknown but are likely to be similar. Chin et al. (2004) estimated that fog results in 3,400,000 VHD on the nation's urban arterial streets.

Research is needed to quantify the effect of fog, dust storms, smoke, and high winds on urban street operation. The research should develop a procedure for quantifying the effect of these weather conditions on speed, saturation flow rate, and other traffic characteristics that influence urban street operation. The research should also develop models for predicting the frequency and duration of the associated weather events. The research results should be suitable for reliability evaluation.

Research to Improve Specific Models

The urban streets reliability methodology was developed using currently available data and research publications. The data were used to calibrate the various models that make up the methodology. Calibration data were also collected in the field when existing data were not available. In some instances, the research team noted that a model's reliability could be improved if additional data were collected or made available through subsequent research. The following paragraphs identify research that targets these specific models in the methodology.

Wet-Pavement Duration

The findings from one research project indicated that the time required for pavement to dry following a rain event is a function of temperature. Drying time was found to decrease with increasing temperature. However, the research did not consider drying time for temperatures below about 60°F. Research is needed to develop a model for predicting pavement drying time for temperatures ranging from −10°F to 100°F. Other factors that influence drying time (e.g., relative humidity, time of day, cloud presence, wind speed, and pavement type) may be considered in developing the model.

Effect of Weather on Signalized Intersection Saturation Flow Rate

A limited amount of research has investigated the effect of weather on saturation flow rate. This research found that weather events (i.e., rain or snow) reduce saturation flow rate, but the amount of reduction appears to vary depending on other, unmeasured factors. For example, the amount of reduction may be influenced by driver experience in (or familiarity with) driving in poor weather. Research is needed to develop a saturation flow rate adjustment factor for the following weather conditions:

- Clear, dry pavement;
- Rain, wet pavement;
- Clear, wet pavement (not raining);
- Snow, snow or ice on pavement; and
- Clear, snow or ice on pavement (not snowing).

The research should quantify the effect of precipitation rate, grade, and temperature on saturation flow rate. Driver familiarity with the listed weather conditions should also be considered, possibly incorporating this effect using surrogates such as the altitude and latitude of the intersection.

Effect of Incident Length on Segment Operation

As described in Chapter 6, several proposed enhancements were developed for urban streets methodology in Chapter 17 of the HCM2010. One is a procedure for quantifying the effect of a mid-segment incident on segment speed and capacity. This procedure does not consider the length of roadway influenced by the incident. An incident that closes a lane for the length of the segment likely has a larger negative effect on operation than one that closes the same lane but for only a few feet along the street. When an incident's influence length is "short," relative to the length of the segment, its proximity to the upstream or downstream signalized intersection may also have an effect on segment operation. Research is needed to quantify the effect of incident length, duration, and location (relative to the adjacent signals) on segment speed and capacity.

Incident Distribution

Research indicates that the distribution of incident frequency varies in a predictable manner by the following categories: street location (i.e., segment or intersection), event type (crash or noncrash), lane location, and severity (i.e., fatal, injury, property damage only, breakdown, debris). However, research also indicates that the distribution proportions vary by region of the country (possibly explained by weather, terrain, income level, and design standards). They may also vary by the facility's degree of recurring congestion and its geometric design (e.g., presence of roadside barrier). Research is needed to develop a model for predicting the distribution of incident frequency that is suitable for nationwide application.

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APPENDIX A

FREEVAL User's Guide

This document provides guidance on using the FREEVAL-RL (FREeway EVALuation–Reliability) computational engine, which implements the freeway reliability analysis methodology developed in SHRP 2 Project L08. FREEVAL-RL is a Microsoft (MS) Excel–based computational engine coded in the Visual Basic for Applications platform. The tool has a graphical user interface to facilitate data entry and navigation through the tool. The core computational engine of the tool is an enhanced version of FREEVAL-2010, which is the computational engine for the freeway facilities methodology in Chapter 10 of the *Highway Capacity Manual 2010* (HCM2010) (Transportation Research Board of the National Academies 2010). Some modifications and enhancements to the core computational engine of FREEVAL-2010 have been made to make the tool and method ready for reliability analysis. These changes have been documented in a separate working paper.

Overview of FREEVAL-2010 Base Method

The base computational engine, FREEVAL-2010, is a computerized, worksheet-based environment designed to faithfully implement the operational analysis computations for undersaturated and oversaturated directional freeway facilities in the HCM2010. It incorporates the freeway segment procedures outlined in Chapters 11, 12, and 13 of the HCM2010 for basic freeway segments, weaving segments, and merge and diverge segments, respectively. It also contains cell transmission model–based algorithms for oversaturated freeway facilities, and it is able to track queue accumulation and dissipation over multiple segments, as well as multiple time periods. This oversaturated flow procedure is a critical requirement for freeway reliability analysis, which is expected to contain many congested scenarios.

The FREEVAL-2010 tool allows a maximum of 70 freeway segments to be analyzed for a maximum duration of up to twenty-four 15-min time intervals (6 h). The engine can generally handle any facility that falls within these temporal and spatial constraints. However, it is highly recommended that the total facility length not exceed 9 to 12 mi to ensure consistency between demand variability over time and facility travel time. Further, the first and last facility segments should be uncongested in the first and last time intervals to allow all queues to form and clear within the facility and study period. This practice assures that the performance measures account for the full extent of congestion and delay. These aspects are discussed in detail in HCM2010, Chapters 10 and 25. In conformance with the HCM2010, all analyses are carried out using U.S. customary units.

FREEVAL-2010 is organized as a sequence of linked MS Excel worksheets and can be used autonomously to analyze individual freeway segments or an entire directional facility. The user must define different freeway segments and enter all necessary input data that are required in the individual segment chapters. These include segment length, number of lanes, length of acceleration and deceleration lanes, heavy and recreational vehicle percentages, and the free-flow speed. The latter can also be calculated in FREEVAL-2010 from the segment or facility geometric attributes.

Consistent with Chapter 10, FREEVAL-2010 covers undersaturated and oversaturated conditions. For oversaturated analysis periods, traffic demands, volume served, and queues are tracked over time and space, as discussed in detail in HCM2010, Chapter 25. In addition to characterizing oversaturated conditions, the most significant difference from the segment-based chapters is that FREEVAL carries out all calculations using 15-min flow rates (expressed in vehicles per hour). It therefore does not use a peak hour factor. To replicate the example problem results found in the segment chapters, peak hour factor–adjusted flow rates must be entered in FREEVAL directly. Heavy-vehicle adjustments (using general terrain factors or directly input for specific grade segments) are automatically handled by the methodology.

The computational engine is further designed to allow the user to revise input data following the completion of an analysis. This feature is intended to perform quick sensitivity or "what if" analyses of different demand scenarios or geometric changes to the facility. However, the user is cautioned to ensure that all prior inputs are maintained when using FREEVAL for extensive scenario evaluation. FREEVAL-2010 is not a commercial software product. It relies on the voluntary commitment of the Transportation Research Board Committee on Highway Capacity and Quality of Service to address software bugs that may emerge in the course of its use and to incorporate methodological changes over time.

The next section focuses on the step-by-step coding process of the FREEVAL-RL tool. The analysis process starts with a basic input of project summary information and continues with a detailed input of the facility seed file in a process similar to FREEVAL-2010, as well as detailed input in the freeway scenario generator (FSG). Each of these components is discussed in detail. The appendix concludes with a discussion of the automated output generated by FREEVAL-RL.

FREEVAL-RL Coding Process

This section provides step-by-step guidance for the use of the FREEVAL reliability (FREEVAL-RL) tool. There are five major steps to run the tool:

1. Project ID information;
2. Seed file management;
3. Scenario management;
4. Running scenarios; and
5. Viewing results.

In the first step, the user inputs general descriptive information about the project. Information gathered in this step is used to name the associated files. In the second step, seed file management, the user can create either a new seed file or view or edit an existing file. This step is conceptually similar to the existing process of coding a freeway facility in the FREEVAL-2010 computational engine, as the user enters detailed geometric and operational characteristics of the seed file. In the third step, FREEVAL-RL invokes the FSG to generate the various scenarios. The FSG is a separate computational engine that is automatically invoked and exchanges information with the FREEVAL-RL tool. In Step 4, the user runs the generated scenarios, which can be a time-intensive process, as many scenarios are automatically executed in a batch-run process. Step 5 generates a summary output report or provides access to a more detailed output matrix file.

Each step is activated by clicking the associated button in the main menu. The main menu can be accessed by clicking Go to The Main Menu in the Intro worksheet of FREEVAL. This button can always be used in the middle of the process to display the FREEVAL-RL main menu (Figure A.1).

The FREEVAL-RL tool requires the use of Visual Basic macros, which may need to be allowed or enabled depending on the settings in the MS Excel program. Those not familiar with MS Excel security options can refer to the MS Excel 2010

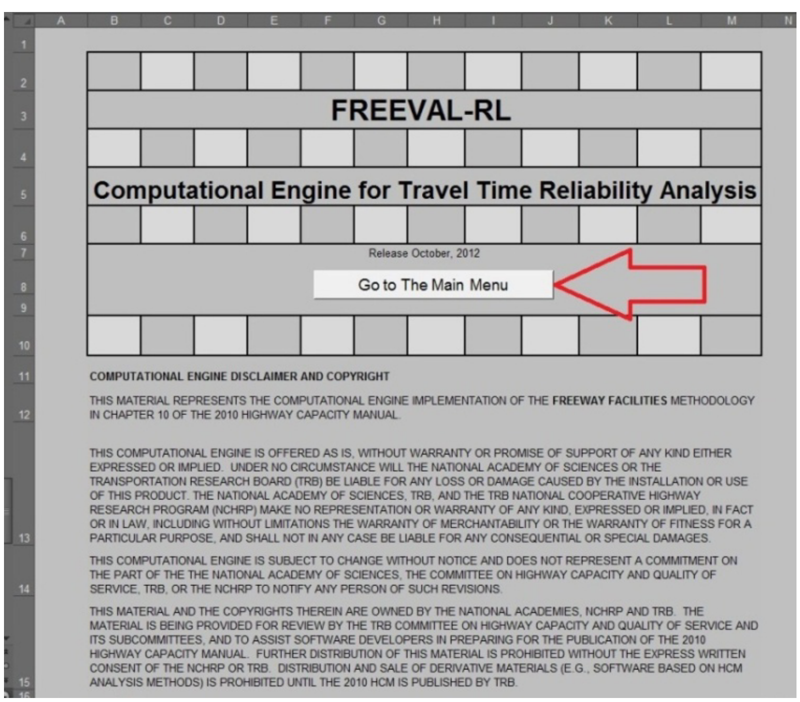


Figure A.1. FREEVAL-RL Intro worksheet.

Security Options Quick Guide of this appendix for quick guidance.

Step 1. Enter Project Summary

The FREEVAL-RL analysis process starts with entering project summary information. Start by clicking Enter Project Summary on the main user form, as shown in Figure A.2.

After selecting the Enter Project Summary button, four input boxes appear (see Figure A.3):

1. File Name Header: This entry will be used as the file header name for all files generated in the project. For example, if the user enters "I-999," all the file names associated with this project that are generated by the computational engine will begin with the string of characters "I-999."
2. Summary Report Header: This entry is depicted as the header in the summary report discussed later in this appendix.
3. Analyst: Information about the analyst.
4. Analysis Date: Date when the analysis is performed.

After entering all desired information, select OK to return to the main menu.

Step 2. Seed File Management

In this second step the user creates, edits, or views the seed file, which is a single, representative FREEVAL input file. Click Create The Seed File to start a new seed file (see Figure A.4). If a seed file has already been created, click View/Edit The Seed File to access the file; this button will be inactive if the seed file has not been created.

If the user elects to create a new seed file, another menu form will appear to collect the necessary information to create

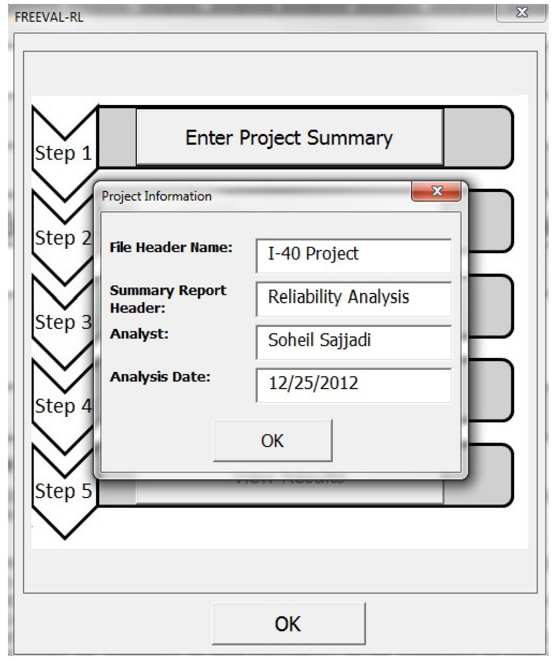


Figure A.3. Step 1, User form.

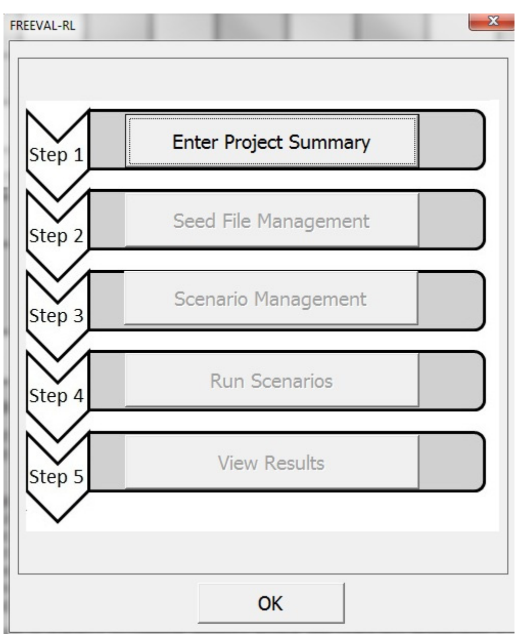


Figure A.2. FREEVAL-RL main user form, Step 1.

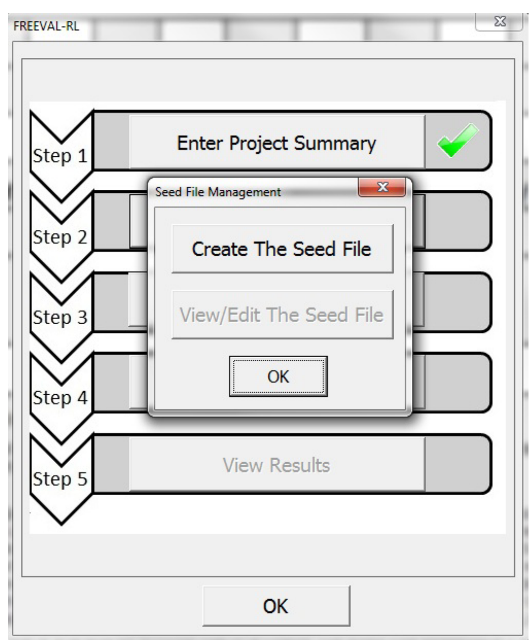


Figure A.4. Seed file management user form.

the seed file. This user form, shown in Figure A.5, requires the following entries:

1. Study Period Start Time: Here, hours and minutes are entered separately, with hours expressed in a 24-h format (e.g., 6 p.m. = 18) and minutes rounded to the nearest 15 min (e.g., 00, 15, 30, or 45).
2. Study Period End Time: Time of day when the analysis ends, expressed in two-digit hour and two-digit minute format (hh:mm).

   Note that the total study period duration cannot exceed 6 h. Also, the start and end times should be for the same calendar day.
3. Analysis Year: The year in which the reliability analysis is performed.
4. RRP Start Date: Reliability reporting period (RRP) start day in two-digit month and two-digit day format (MM/DD).
5. RRP End Date: RRP end date in two-digit month and two-digit day format (MM/DD).

   Note that the RRP end date must follow the RRP start date in the same year.
6. Seed Demand Day in RRP: The date represented by the demand volumes used in the seed file is entered. Similar to other RRP-related information, this date should be entered in the two-digit month and two-digit day format (MM/DD).

   Note that the seed demand date must be able to be extrapolated to the RRP if they do not overlap. Say, for example, the RRP consists of the summer months only, and the seed data were collected in the winter months. Appropriate demand adjustment factors (DAFs) from winter to summer must be made available to the analyst in order to be able to reconstruct the summer demand patterns.
7. Number of HCM Segments: Total number of facility HCM segments is entered in this field.
8. Terrain: Please consult the FREEVAL-2010 user guide for exercising this option.
9. Ramp Metering: Please consult the FREEVAL-2010 user guide for exercising this option.
10. Jam Density: Proposed jam density of the facility is selected in this combo box.
11. Capacity Drop in the Queue Discharge Mode (%): This is a recent enhancement to the HCM2010 freeway facilities methodology. It indicates the percentage capacity drop when operating the queue discharge mode after breakdown (demand/capacity > 1). This range varies from 0% to 10%.

After this step, the structure of the seed file will be configured permanently. Therefore, in the preceding input box the user must confirm that the provided information is final in order to proceed with the current input settings (see Figure A.6).

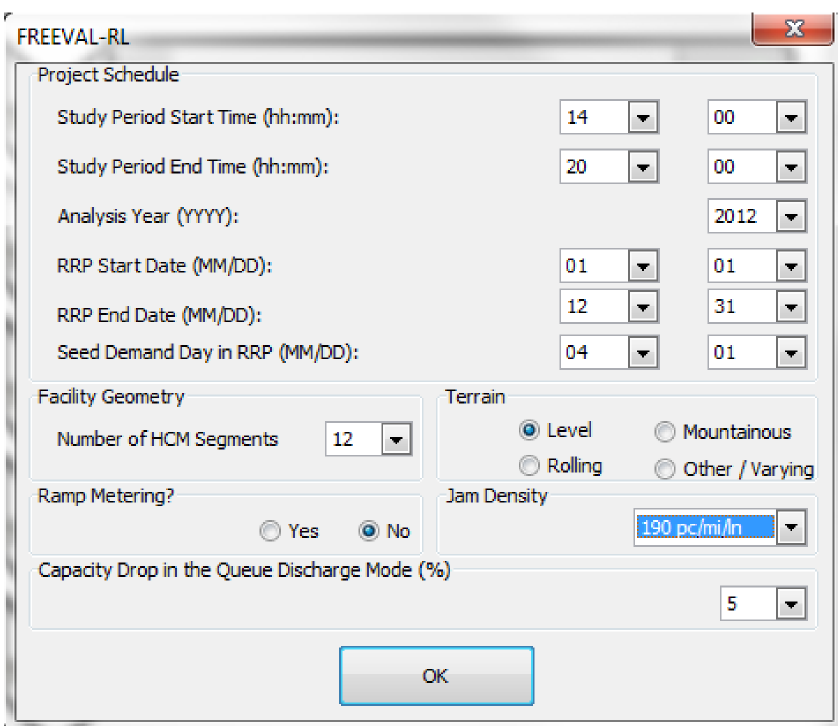


Figure A.5. Seed file creation user form.

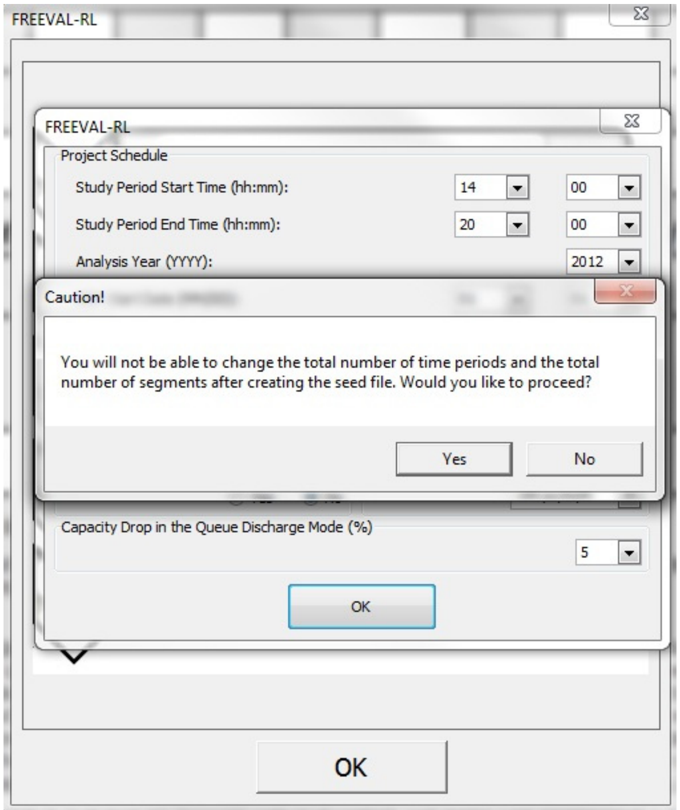


Figure A.6. Data entry verification input box.

After the user clicks Yes in the input box, as shown in Figure A.6, the seed file is created using the total number of HCM segments and analysis (i.e., 15-min) periods. At this stage, the user enters the detailed facility data for the different analysis periods. In order to complete seed file data entry, two substeps are required related to coding the segment type and segment data entry. These substeps (see Figure A.7) are explained in detail in the following sections.

Step 2A. Code Segment Types

Here the user enters each HCM segment type (see Figure A.8). Note that the number of columns has been reduced to match the number of segments defined by the user. The proper way to define the appropriate number of segments is explained in HCM2010, Chapter 10, including the requirement that the first and last segments of the facility should be coded as basic segments. The number of input worksheets generated matches the number of (15-min) analysis periods entered earlier. Using drop-down menus, the user defines each segment as a basic, on-ramp, off-ramp, weaving, or overlapping ramp segment following HCM conventions (see HCM2010, Chapter 10). After identifying all segment types, the user clicks the Step 2-A: Segment Types Entered button. After this action, a macro will automatically black out all unneeded data entry cells.

Step 2B. Segment Data Entry

Next, the user enters data for each segment and each analysis period in sequence (see Figure A.9). The common inputs needed for all segments are length (feet), number of lanes, free-flow speed (miles per hour), segment demand (number of vehicles per hour), percentage trucks, and percentage recreational vehicles. The user can use several adjustment factors that may affect the operations of the facility. These factors are discussed in a later section. For all ramp and weaving segments, the user further needs to enter the ramp demand flows and can adjust the heavy-vehicle percentages as desired. An analysis period corresponds to a 15-min period, and as a result all volume inputs should be in the form of 15-min demand flow rates (in vehicles per hour). No peak hour factor adjustment is necessary.

After entering all input for the first analysis period, the user proceeds to the remaining analysis periods to enter the corresponding input data. For all subsequent analysis periods, some inputs are automatically copied from the "t = 1" worksheet. However, the engine generally allows the user to override these automatically generated entries. Demand volumes always need to be entered for all analysis periods. After completing all inputs for all analysis periods and checking for correctness, the user clicks Run The Seed File/Go to The Main Menu.

If this is the first time the seed file has been run, clicking the Run The Seed File/Go to The Main Menu button will

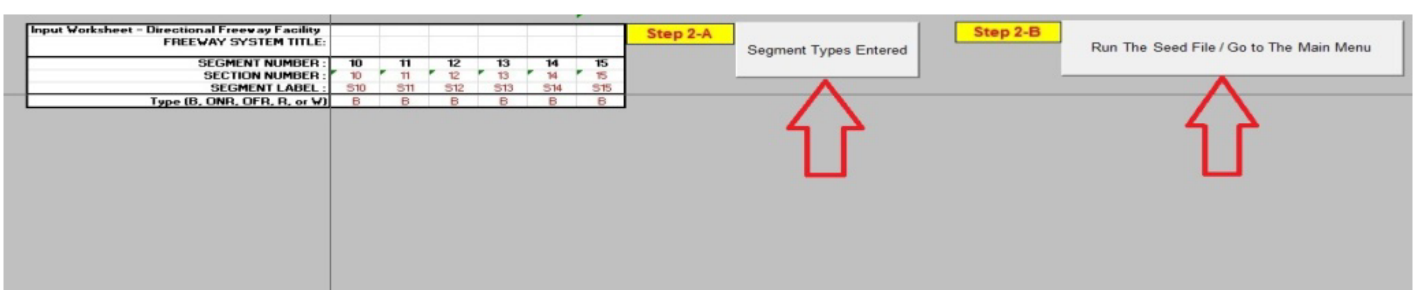


Figure A.7. Step 2 substeps: Step 2A (segment type coding) and Step 2B (segment data entry).

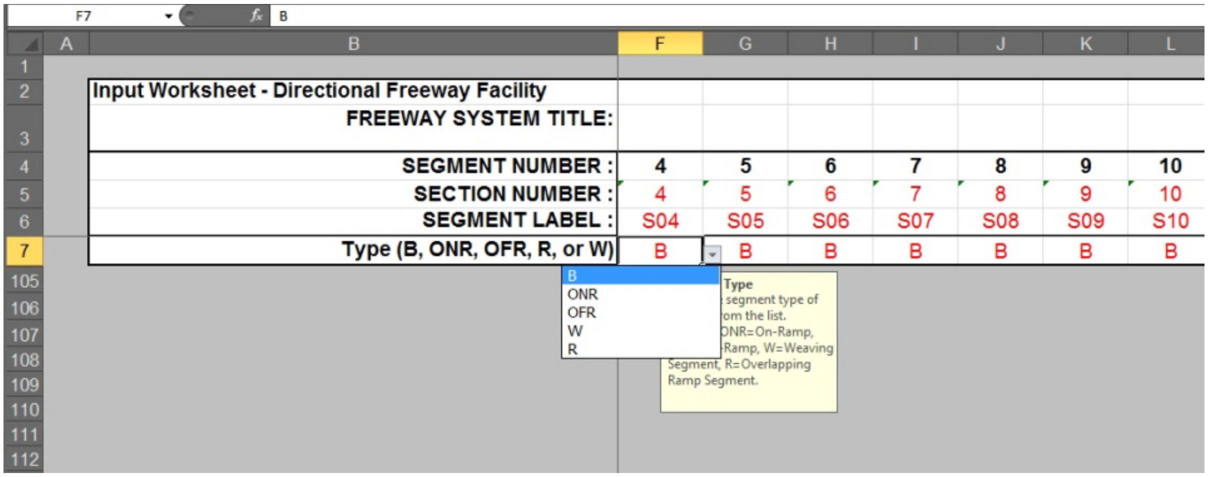


Figure A.8. Step 2A: entering facility segment types.

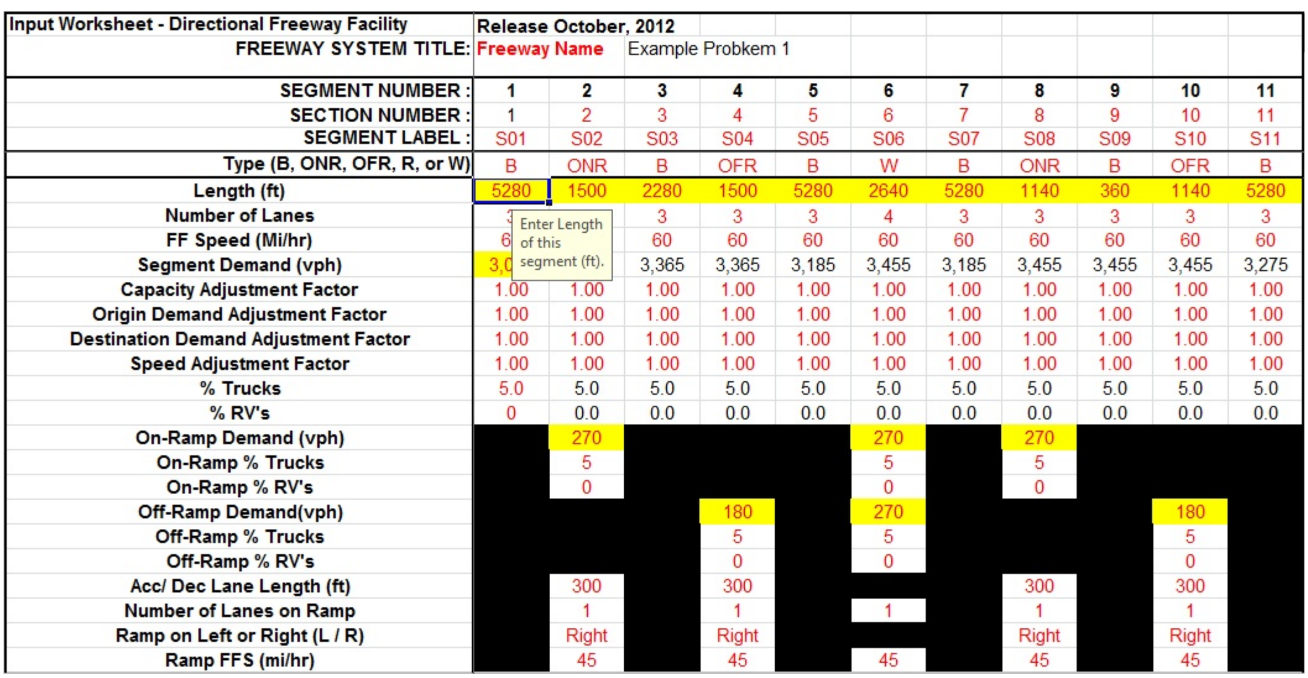
automatically execute the seed file, after which the main menu user form will be displayed. If the seed file has already been run (e.g., View Mode), the main menu is opened without running the seed file. The seed file is automatically saved when the run process is finished, using as a header the string specified in the initial project information dialog box.

For more information regarding how to code any HCM2010 seed file and for detailed interpretation of the output, refer to the FREEVAL-2010 user guide available in Volume 4 of the HCM2010.

Optional Step. Revise Input Data

As an optional step, the user can revise inputs in the seed file by clicking View/Edit The Seed File in the second step and clicking Revise Input Data in the Results Summary worksheet (see Figure A.10).

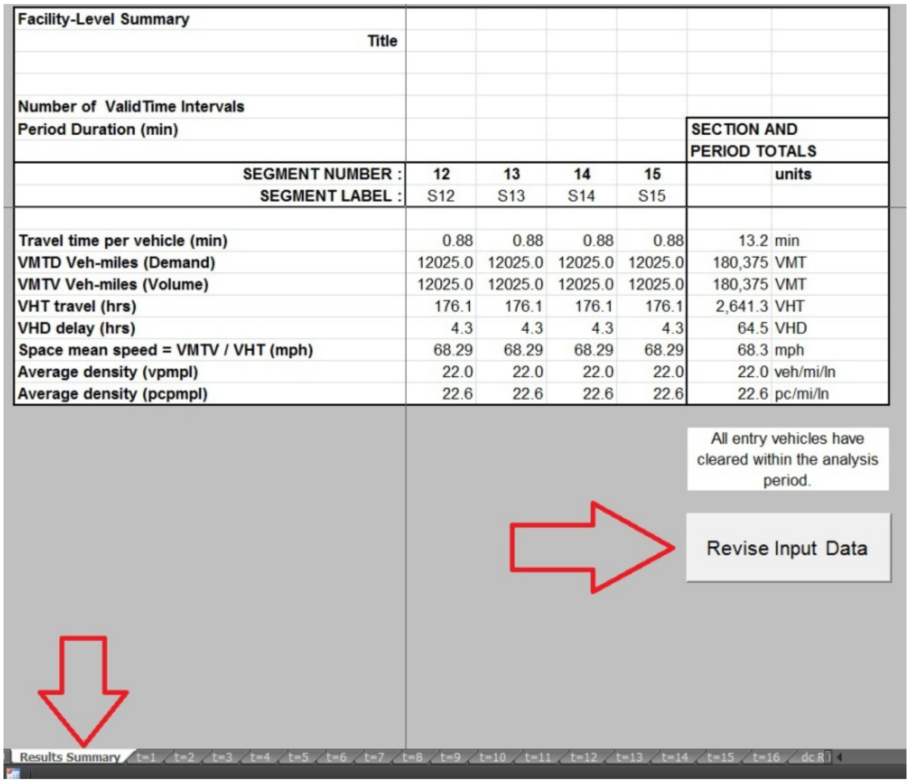
Clicking Revise Input Data opens a dialog box similar to Create The Seed File user form. Please use caution in this step because the total number of time periods and number of HCM segments should definitely remain fixed (see Figure A.11).



Input Worksheet - Directional Freeway Facility	Release October, 2012										
FREEWAY SYSTEM TITLE:	Freeway Name		Example Probkem 1								
SEGMENT NUMBER :	1	2	3	4	5	6	7	8	9	10	11
SECTION NUMBER :	1	2	3	4	5	6	7	8	9	10	11
SEGMENT LABEL :	S01	S02	S03	S04	S05	S06	S07	S08	S09	S10	S11
Type (B, ONR, OFR, R, or W)	B	ONR	B	OFR	B	W	B	ONR	B	OFR	B
Length (ft)	5280	1500	2280	1500	5280	2640	5280	1140	360	1140	5280
Number of Lanes	3	[illegible]	3	3	3	4	3	3	3	3	3
FF Speed (Mi/hr)	6	[illegible]	60	60	60	60	60	60	60	60	60
Segment Demand (vph)	3,0	[illegible]	3,365	3,365	3,185	3,455	3,185	3,455	3,455	3,455	3,275
Capacity Adjustment Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Origin Demand Adjustment Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Destination Demand Adjustment Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Speed Adjustment Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
% Trucks	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
% RV's	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
On-Ramp Demand (vph)		270				270		270			
On-Ramp % Trucks		5				5		5			
On-Ramp % RV's		0				0		0			
Off-Ramp Demand(vph)				180		270				180	
Off-Ramp % Trucks				5		5				5	
Off-Ramp % RV's				0		0				0	
Acc/ Dec Lane Length (ft)		300		300				300		300	
Number of Lanes on Ramp		1		1		1		1		1	
Ramp on Left or Right (L / R)		Right		Right				Right		Right	
Ramp FFS (mi/hr)		45		45		45		45		45	

Enter Length of this segment (ft).

Figure A.9. Step 2B: segment data entry.



Facility-Level Summary						
Title						
Number of ValidTime Intervals						
Period Duration (min)					SECTION AND PERIOD TOTALS	
SEGMENT NUMBER :	12	13	14	15		units
SEGMENT LABEL :	S12	S13	S14	S15		
Travel time per vehicle (min)	0.88	0.88	0.88	0.88	13.2	min
VMTD Veh-miles (Demand)	12025.0	12025.0	12025.0	12025.0	180,375	VMT
VMTV Veh-miles (Volume)	12025.0	12025.0	12025.0	12025.0	180,375	VMT
VHT travel (hrs)	176.1	176.1	176.1	176.1	2,641.3	VHT
VHD delay (hrs)	4.3	4.3	4.3	4.3	64.5	VHD
Space mean speed = VMTV / VHT (mph)	68.29	68.29	68.29	68.29	68.3	mph
Average density (vpmpl)	22.0	22.0	22.0	22.0	22.0	veh/mi/ln
Average density (pcpmpl)	22.6	22.6	22.6	22.6	22.6	pc/mi/ln

Figure A.10. Revise input data.

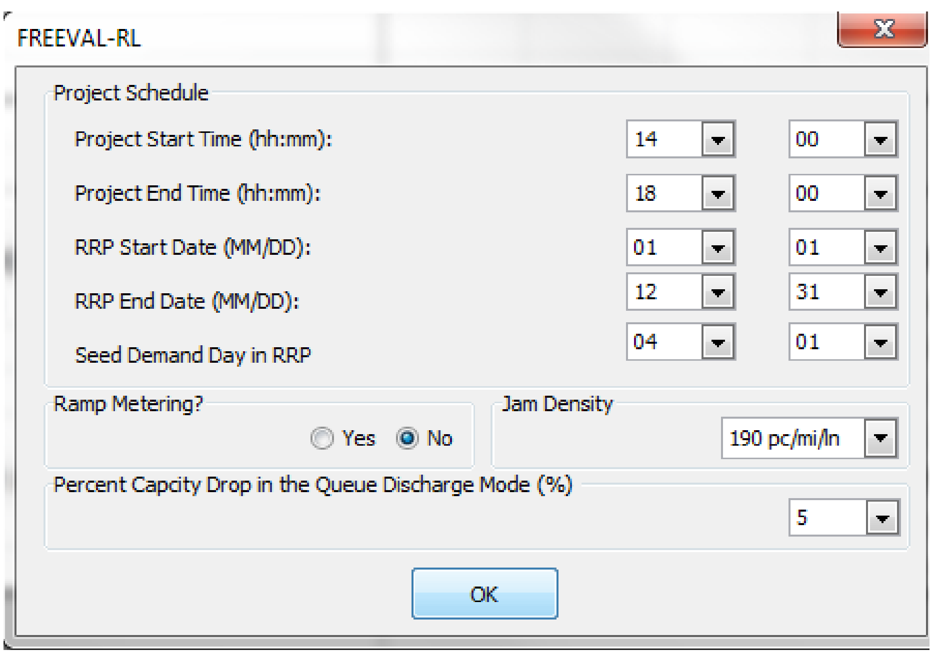


Figure A.11. Revise input data user form.

Step 3. Scenario Management

The goal of this section is to provide guidance for users who wish to generate scenarios for reliability analysis. Scenario generation is the third step in the reliability analysis using FREEVAL-RL.

By clicking the Scenarios Management button in the FREEVAL-RL main menu, the user is prompted to locate the FSG file. This separate MS Excel file should be located within the same working directory as the FREEVAL-RL file. Separate copies of FREEVAL-RL and FSG can be saved in separate folders for each reliability analysis.

The user either can open an empty FSG file or use an earlier version (in which the file name starts with the project name) that has been previously customized for a facility. When an appropriate FSG file is selected, the user is directed to the FSG file by clicking Open.

The FSG process consists of five steps in which the user enters the different types of information needed to generate the recurring and nonrecurring congestion scenarios for FREEVAL-RL on the following worksheets:

1. Start worksheet;
2. Demand pattern worksheets;
3. Weather probability worksheet;
4. Incident probability worksheet; and
5. Detailed scenario worksheet.

The five steps should be followed in order; otherwise, the scenario generation process could fail. Each step is discussed in detail.

Step 3A. Start Worksheet

As a first step, the user must locate the appropriate seed file for the FSG to extract the necessary information for developing and generating scenarios. Click Step 1: Read Seed File in the Start worksheet. The seed file is the FREEVAL-RL file that the user has created in the previous steps. Figure A.12 shows the schematic of the Start worksheet. As a general rule, in all FSG worksheets yellow-highlighted cells represent input data cells that can be entered or altered by the user. Proceed to the next step by clicking Step 2: Demand Pattern Configuration.

Step 3B. Demand Pattern Worksheets

In this step, the time-dependent demand patterns are defined in the RRP. It consists of two worksheets. In the first, the

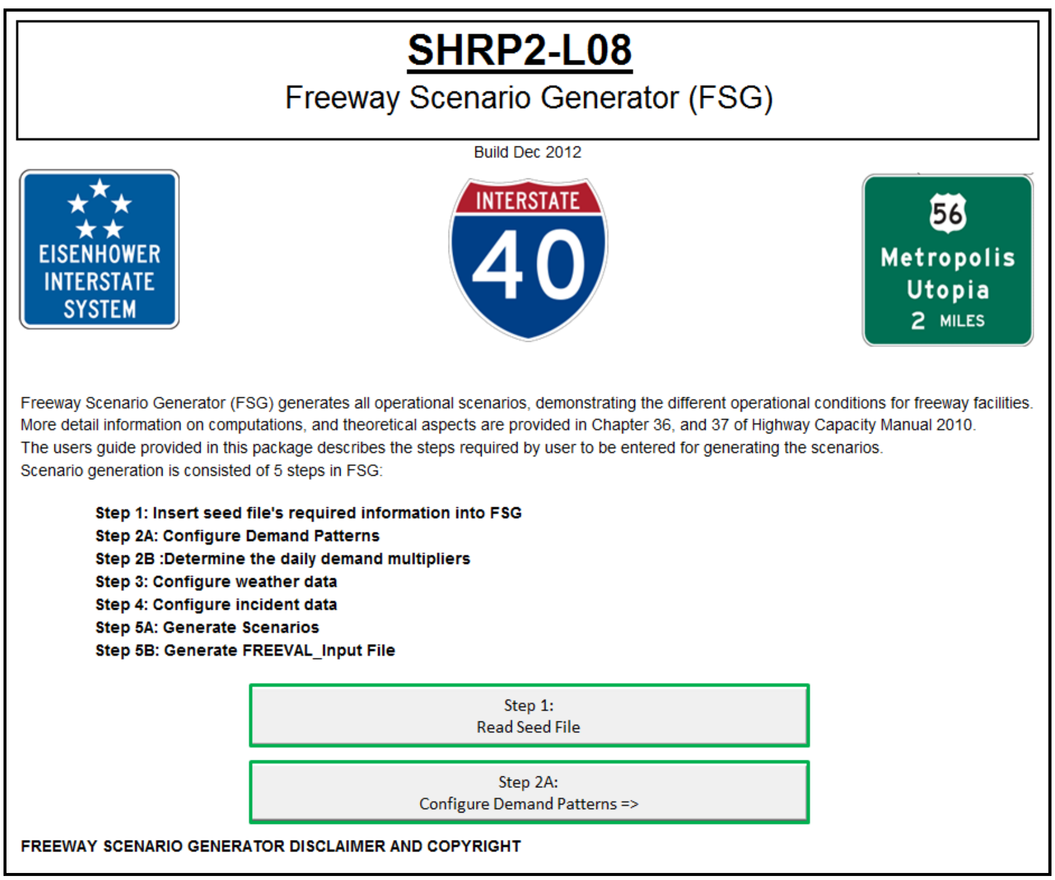


Figure A.12. Schematic of the Start worksheet in the FSG.

overall demand pattern input is displayed in a calendar format to show the configuration of demand patterns for the subject facility. In this step, the user configures similar seasons, months, and weekdays that will be combined within the same demand pattern. In the second worksheet, the user can configure the daily and monthly DAFs based on daily and monthly variability of traffic demand for the subject facility or by using national defaults for urban and rural freeways.

Step 3B.1. Demand Pattern Configuration Worksheet

If no demand pattern has been coded previously, then the calendar at the middle of the screen will be empty. Figure A.13 shows a previously coded Demand Pattern worksheet. Each number in parentheses (and cell color) following a calendar date represents a unique demand pattern by day of week and month of year to be analyzed. Time-of-day demand variations have already been incorporated as input into the seed file in various 15-min analysis periods.

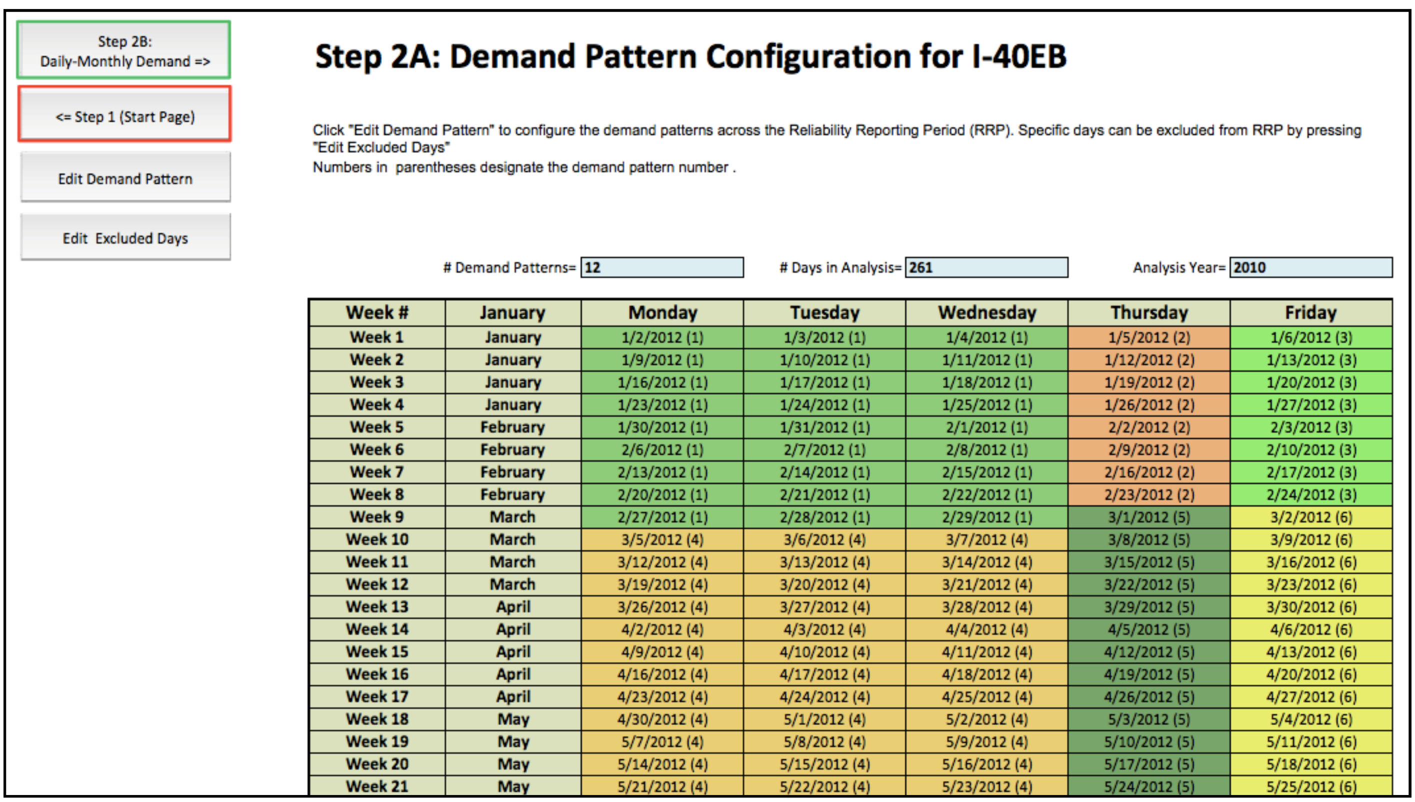
By clicking the Edit Demand Pattern button on the left of the screen, the user is directed to a form that enables the definition or redefinition of the different demand patterns, as shown in Figure A.14. This form consists of two sections. The left portion is for configuring the demand patterns across days of the week; the right section is for configuring the demand patterns across months of the year.

To define a demand pattern, ordinal numbers starting from 1 are used to assign each day to a demand pattern. For example, if all weekdays are assumed to have the same demand pattern, the value "1" is entered for each weekday. If, on the other hand, Mondays, Tuesdays through Thursdays, and Fridays have different patterns, then 1 is entered for Mondays; 2 for Tuesdays, Wednesdays, and Thursdays; and 3 for Fridays, as shown in Figure A.14.

On the right side of the form, the same data entry logic applies for designating demand variability across months. Here, the user can combine different months of the year into the same demand pattern. Defaults are available for using a different pattern for each month or for combining months into four seasons or two seasons. The selection of daily and monthly demand pattern combination should be informed by local traffic data on the subject facility.

By clicking the Apply Default Demand Pattern button located on the upper section of the form, appropriate demand patterns based on national defaults are inserted.

By clicking Accept Demand Pattern and Continue, the user is directed back to the Step 2A worksheet. The user can now exclude specific days (e.g., holidays or special event days) from the analysis by clicking Edit Excluded Days in the Demand Pattern worksheet. The Add or Delete Excluded Days form pops up for the user to enter calendar dates to be excluded from the reliability calculations (see Figure A.15). If an error



Step 2B: Daily-Monthly Demand =>

<= Step 1 (Start Page)

Edit Demand Pattern

Edit Excluded Days

Step 2A: Demand Pattern Configuration for I-40EB

Click "Edit Demand Pattern" to configure the demand patterns across the Reliability Reporting Period (RRP). Specific days can be excluded from RRP by pressing "Edit Excluded Days"
Numbers in parentheses designate the demand pattern number .

Demand Patterns= 12 # Days in Analysis= 261 Analysis Year= 2010

Week #	January	Monday	Tuesday	Wednesday	Thursday	Friday
Week 1	January	1/2/2012 (1)	1/3/2012 (1)	1/4/2012 (1)	1/5/2012 (2)	1/6/2012 (3)
Week 2	January	1/9/2012 (1)	1/10/2012 (1)	1/11/2012 (1)	1/12/2012 (2)	1/13/2012 (3)
Week 3	January	1/16/2012 (1)	1/17/2012 (1)	1/18/2012 (1)	1/19/2012 (2)	1/20/2012 (3)
Week 4	January	1/23/2012 (1)	1/24/2012 (1)	1/25/2012 (1)	1/26/2012 (2)	1/27/2012 (3)
Week 5	February	1/30/2012 (1)	1/31/2012 (1)	2/1/2012 (1)	2/2/2012 (2)	2/3/2012 (3)
Week 6	February	2/6/2012 (1)	2/7/2012 (1)	2/8/2012 (1)	2/9/2012 (2)	2/10/2012 (3)
Week 7	February	2/13/2012 (1)	2/14/2012 (1)	2/15/2012 (1)	2/16/2012 (2)	2/17/2012 (3)
Week 8	February	2/20/2012 (1)	2/21/2012 (1)	2/22/2012 (1)	2/23/2012 (2)	2/24/2012 (3)
Week 9	March	2/27/2012 (1)	2/28/2012 (1)	2/29/2012 (1)	3/1/2012 (5)	3/2/2012 (6)
Week 10	March	3/5/2012 (4)	3/6/2012 (4)	3/7/2012 (4)	3/8/2012 (5)	3/9/2012 (6)
Week 11	March	3/12/2012 (4)	3/13/2012 (4)	3/14/2012 (4)	3/15/2012 (5)	3/16/2012 (6)
Week 12	March	3/19/2012 (4)	3/20/2012 (4)	3/21/2012 (4)	3/22/2012 (5)	3/23/2012 (6)
Week 13	April	3/26/2012 (4)	3/27/2012 (4)	3/28/2012 (4)	3/29/2012 (5)	3/30/2012 (6)
Week 14	April	4/2/2012 (4)	4/3/2012 (4)	4/4/2012 (4)	4/5/2012 (5)	4/6/2012 (6)
Week 15	April	4/9/2012 (4)	4/10/2012 (4)	4/11/2012 (4)	4/12/2012 (5)	4/13/2012 (6)
Week 16	April	4/16/2012 (4)	4/17/2012 (4)	4/18/2012 (4)	4/19/2012 (5)	4/20/2012 (6)
Week 17	April	4/23/2012 (4)	4/24/2012 (4)	4/25/2012 (4)	4/26/2012 (5)	4/27/2012 (6)
Week 18	May	4/30/2012 (4)	5/1/2012 (4)	5/2/2012 (4)	5/3/2012 (5)	5/4/2012 (6)
Week 19	May	5/7/2012 (4)	5/8/2012 (4)	5/9/2012 (4)	5/10/2012 (5)	5/11/2012 (6)
Week 20	May	5/14/2012 (4)	5/15/2012 (4)	5/16/2012 (4)	5/17/2012 (5)	5/18/2012 (6)
Week 21	May	5/21/2012 (4)	5/22/2012 (4)	5/23/2012 (4)	5/24/2012 (5)	5/25/2012 (6)

Figure A.13. Demand Pattern worksheet in the FSG.

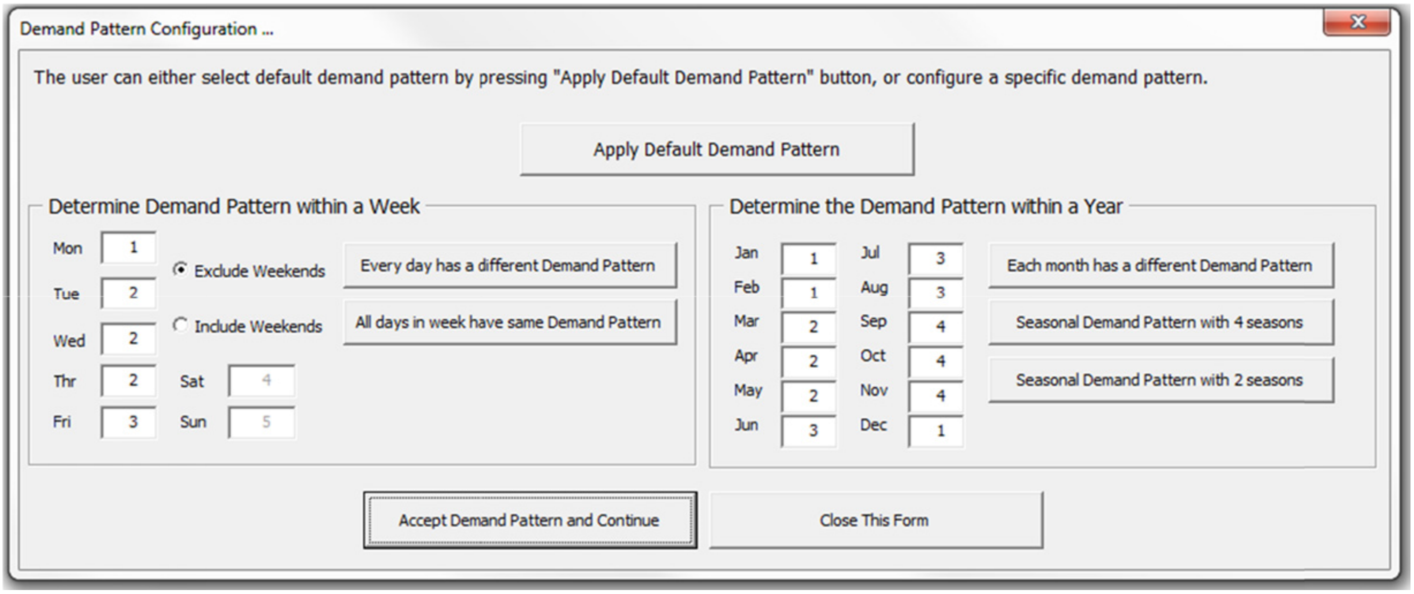


Figure A.14. Demand Pattern configuration form in the FSG.

is made, a day can be added using the Add Excluded Day functionality.

Step 3B.2. Demand Weekday-Month Worksheet

The user can assign demand adjustments (called multipliers) for the facility in the table provided in the weekday-month demand multiplier worksheet. All adjustments in this table are based on the ratio of the cell value to the annual average daily traffic (AADT) for the facility being analyzed. If such values do not exist locally, then the user can select tabulated national default values for either urban or rural freeway facilities. In the implementation of the method, the demand for each scenario is adjusted from the seed file values.

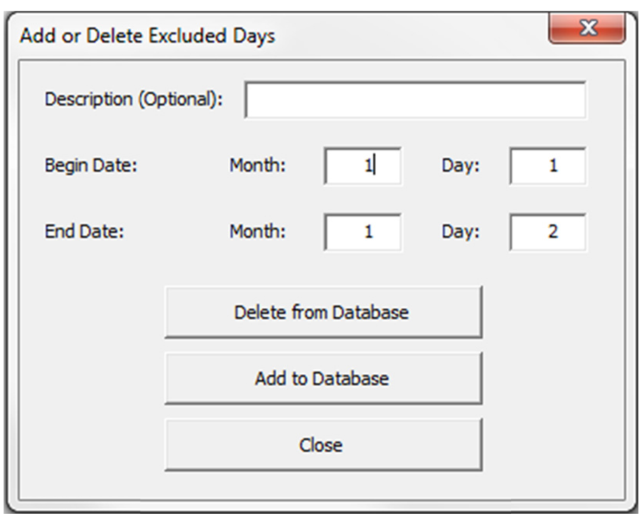


Figure A.15. Add or Delete Excluded Days form in the FSG.

This process is best explained using a numerical illustration. Say the user coded in the seed file traffic demands that represent Thursdays in January. Figure A.16 shows that Thursdays in January have a demand multiplier equal to 1.052. This means that the seed file demands are 5.2% higher than the AADT. To generate demand volumes for Fridays in the spring (Demand Pattern 6 in Figure A.13), for which the multiplier is computed at 1.198 (a weighted average based on the highlighted numbers in Figure A.16), the demand volumes for each analysis period in the seed file are multiplied by the ratio 1.198/1.052, or 1.139. It goes without saying that for demand patterns with multipliers below 1.052 (e.g., Mondays, Tuesdays, and Wednesdays in the fall in Figure A.16), the resulting ratio would be less than 1.0.

Finally, by clicking Insert Facility Specific, the table will be cleared and the user can enter locally derived demand multipliers for the facility.

Step 3C. Weather Probability Worksheet

This worksheet, which requires four categories of information, is designed to capture all necessary weather information for the generation of scenarios that include weather events and their impacts. Figure A.17 depicts a sample weather worksheet screenshot with data shown for Raleigh, North Carolina.

The upper table in Figure A.17 shows the temporal probabilities for different weather categories for each month. Each cell represents the ratio of the number of hours in which a weather event occurred divided by the number of hours in the study periods falling in each month. For example, the 5.911% value (light snow in January) represents the ratio (percentage)

Step 2:
Weather = >

Step 1A:
<= Demand Pattern

Insert Urban Default Values

Insert Rural Default Values

Insert Facility Specific

Insert One of availble Sites

Step 2B: Daily Demand Multipliers for I-40EB

The Analyst can either accept the default demand multipliers (DM) for each combination of weekdays and months of year, or insert facility-specific factors if available. (Reference is to facility AADT)

DM		Day of Week				
		Monday	Tuesday	Wednesday	Thursday	Friday
Month	January	0.996623	1.027775	1.040394	1.052601	1.081612
	February	0.939253	1.010728	1.039214	1.092029	1.140072
	March	1.043305	1.069335	1.063524	1.110921	1.171121
	April	1.073578	1.087455	1.098238	1.161974	1.215002
	May	1.076331	1.106182	1.113955	1.157717	1.210434
	June	1.078043	1.085853	1.067470	1.138720	1.180327
	July	1.082580	1.070993	1.102512	1.147279	1.184981
	August	1.046045	1.052146	1.060371	1.093243	1.164901
	September	1.016023	1.024051	1.023625	1.074782	1.152946
	October	1.048981	1.045723	1.066986	1.107044	1.160954
	November	0.974044	0.999947	1.041211	1.081541	1.070354
	December	0.974785	0.956475	0.987019	0.916107	1.007695

Figure A.16. Demand weekday-month worksheet in the FSG.

of hours in the 2 to 8 p.m. study period in which light snow occurred in January in Raleigh to the total number of hours in January between 2 and 8 p.m. These estimates are based on 10 years of meteorological historical data extracted for 101 metropolitan areas in the United States. The user can directly download the probabilities by clicking the Raleigh, NC button shown in Figure A.17, selecting the metro area from the pull-down menu, and then clicking Extract Long Term Regional Weather Data for Specified Location. The user can, of course, override any cell values and directly enter facility-specific weather probabilities when those are available.

The lower table documents the key operational characteristics of each weather category. The first row pertains to the mean duration of each weather type with respect to the location of the facility. If the FSG weather database is selected to fill the upper table (for the nearest metropolitan area), then the mean duration for weather types for the selected metropolitan area will be automatically filled in the first row.

The second and third rows in the lower table require weather event inputs for the capacity adjustment factor (CAF) and free-flow speed adjustment factor (SAF), respectively. These factors are used in FREEVAL-RL to model weather event

Step 3:
Incidents =>

Step 1B:
<= Demand Day-Month

Insert Facility Specific

Step 3: Weather Data for I-40EB

The analyst can either use weather defaults based on 10 year average by pressing "Extract Longterm Regional Weather Data for Specified Location", or directly put the probabilities for different weather categories. If the user wants to use 10 year average, then he needs to select the nearest metropolitan area to the facility.

Select Nearest Metro Politan area to the Facility	Raleigh, NC

Extract Longterm Regional Weather
Data for Specified Location

	Weather Categories (based on HCM 2010 Chapter 10: Freeway Facilities)										
Month	Med Rain	Heavy Rain	Light Snow	LM Snow	MH Snow	Heavy Snow	Severe Cold	Low Vis	Very Low Vis	Min Vis	Normal Weather
January	1.970%	0.000%	5.911%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	92.1182%
February	2.717%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	2.174%	0.000%	0.000%	95.1087%
March	0.505%	0.000%	1.010%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	98.4848%
April	0.000%	0.543%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	99.4565%
May	1.951%	1.951%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	96.0976%
June	0.505%	0.505%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	98.9899%
July	0.500%	0.500%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	99.0000%
August	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	100.0000%
September	4.255%	0.532%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	95.2128%
October	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	100.0000%
November	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	100.0000%
December	0.000%	0.000%	7.805%	0.488%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	91.7073%

Average Duration for Weather Type(min):	42.9	31.0	134.3	46.6	25.8	5.5	15.0	57.2	15.0	136.5	
Default Capacity Adjustment Factor:	92.76%	85.87%	95.71%	91.34%	88.96%	77.57%	91.55%	90.33%	88.33%	89.51%	100.00%
Default FFS Adjustment Factor:	93.00%	92.00%	87.00%	86.00%	84.00%	83.00%	93.00%	94.00%	92.00%	92.00%	100.00%
Demand Adjustment Factor	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	50.00%	100.00%

Figure A.17. Weather worksheet in the FSG.

impacts on facility operations. The CAF and SAF tables are currently filled with national default values in the HCM2010, but these values can be overridden by the user. The last row in the lower table enables the specification of a DAF for each weather category. There are no default values here, but local conditions may provide guidance as to the level of demand adjustments associated with the more severe weather conditions. For example, the analyst may code a DAF less than 1.0 for a heavy snow event to reflect the fact that many drivers may avoid travel during those severe-weather events.

Step 3D. Incident Probability Worksheet

The incident probability worksheet characterizes incident events in terms of probability of occurrence, duration, and severity on the freeway facility. The worksheet is divided into two main sections. The first section, Option A, pertains to those cases for which incident logs are not available or are of insufficient quality to enable direct calculation and entry of incident probabilities. The second section, Option B, allows the user to code facility-specific incident probability data if available. Conceptually, both options result in a table of incident probabilities by incident type and by month of the year. But while Option B requires a data-rich environment with detailed records of incidents, Option A is available for any facility, including ones with no incident data at all.

Option A: Data-Poor Incident Probability Estimation

The intent of Option A is to estimate incident probabilities for a facility with little or no incident field data available. Figure A.18 shows the upper portion of the Option A section of the incident worksheet; the lower portion is identical with Option B, which is discussed next.

Option A is used when the analyst has little or no incident data. In order to estimate these incident probabilities, three different paths are available. The first path is to determine if incident rates (per 100 million vehicle miles traveled [VMT]) are available for the facility. If so, then those rates can be entered on a monthly basis. Allowance is made to enable the user to vary incident rates per month, if such information is available. If not, the second path is to use monthly crash rates for the facility, which are easier to collect, and then use an estimated local incident-to-crash ratio to estimate monthly incident rates. A default factor of the ratio of incidents to crash rates is provided, but it can be overridden by the user.

Option A: for Data Poor Facilities:

Step 1: Enter incident or crash rate (Per 100 million vehicle mile):

Entry data are : ○ Incident Rates ○ Crash Rates ◉ HERS Model

IF monthly crash rates will be entered below then enter site specific crash to incident rate ratio	**6**	National Default Ratio is 4.9

Enter incident or crash rates if known (or use 'HERS model' below the table to estimate crash rate)

Month	Rate
January	152.6
February	152.6
March	152.6
April	152.6
May	152.6
June	152.6
July	152.6
August	152.6
September	152.6
October	152.6
November	152.6
December	152.6

If local crash rates are not availble, Press "Calculate Crash Rate Based on HERS Model" to estimate the crash rate from National Defaults. This requires entry of percent AADT that falls in the Study Period (SP).

Please enter Percent of AADT in the Study Period:	38.84%

Calculate Crash Rate Based on HERS Model

Figure A.18. First (upper) portion of Option A section of the incident worksheet.

If neither crash nor incident rate data are available, then a third path is to generate crash rates using the Highway Economic Requirements System (HERS) model. This option is available by clicking the Calculate Crash Rate Based on HERS Model button shown at the bottom of Figure A.18. If the HERS model is used to generate the crash rates, the user must first provide input on the portion of the AADT that occurs in the typical study period. This input is readily calculated from a known distribution of hourly factors.

This worksheet is designed in a flexible way that adjusts the probability estimation process so that it is compatible with the user's available data. When a path is selected (assume using the HERS model), then the FSG changes the background colors of the appropriate cells so that the user can identify the cells that need data entry.

Figure A.19 presents the remaining input data portion of Option A in the incident worksheet.

The distribution of different incident types along with their mean duration and standard deviation are entered as shown in Figure A.19. Note that the sum of the percentages should add up to 100%. If the distribution of incident types is not available, the user can select national default values in the fourth table by clicking Insert National Default Data.

The temporal probabilities of different incident types are then automatically generated by clicking the Calculate Incident Probabilities button at the bottom of the Option A section of the incident worksheet. The resulting tables are shown in Figure A.20.

Option B: Direct Data Entry for Data-Rich Facilities

As an alternative to estimating incident probabilities, the user can directly enter the monthly incident probabilities by type in the table shown in Figure A.20 (the Option B section of the incident worksheet). If no field data are available, then Option A above should be used to complete this table.

The first table (upper) in Figure A.20 contains month-by-month incident probabilities for six incident types (no incident, shoulder closure, one-lane closure, two-lane closure, three-lane closure, and four-lane closure). Clearly, not all incident configurations are feasible for all facilities, and some may automatically be ignored (e.g., four-lane closures on a three-lane-per-direction facility). In that case, the FSG automatically reallocates the specified probability to the next category of lower severity. Note that the FREEVAL-RL tool does not allow a full facility closure.

The bottom table in Figure A.20 enables entries for incident-specific CAFs per open lane of the freeway. This entry should be determined for different incident types and based on the number of lanes available on the facility. A note of caution: the CAFs shown in Figure A.20 include the frictional effect of the incident impact only. They do not account for the capacity loss due to lane closure. For example, with a single-lane closure on a two-lane segment, the total capacity available would be $0.50 \times 0.7 = 35\%$ of the initial segment capacity. Users should also enter the SAF and the expected DAFs based on local conditions. There are no national default values for either parameter at this time.

In the CAF, SAF, and DAF tables, all unnecessary cells are blacked out. The user only needs to provide (optional) information for cells with a yellow background.

By completing the steps in the incident worksheet, all the necessary information has been entered to produce the scenarios. For this purpose, the user is directed to the Detailed Scenario worksheet to generate the scenarios.

Step 3E. Detailed Scenario Worksheet

In the final step, the various input data from the previous steps are used to generate analysis scenarios. A scenario is a

Step 2: Enter expected duration and standard deviation of the incident types, and their distribution. Incidents will only be moleded when at least one lane is open, otherwise they will be ignored.

Insert National Default Data

Incident Type	Incident Type Distribution	Expected Duration (min)	Std. Dev. of Duration
Shoulder Closure	75.40%	32	15
One Lane Closure	19.60%	34	14
Two Lane Closure	3.10%	53	14
Three Lane Closure	1.90%	69	22
Four Lane Closure	0.00%	69	22
Sum (Check)	100.00%		

Step 3: Press below to Calculate the Probability of Different Incident Types:

Calculate Incident Probabilities

Figure A.19. Second (lower) portion of Option A section in the incident worksheet.

Option B (for Data Rich Facilities): Enter Probabilities Directly In the Table from Incident Logs

By pressing button below, the table clears and analyst can enter facility specific incident probabilities. The incidents will only be modeled where there is at least one lane open, otherwise the scenarios will be ignored.

Insert Facility Specific

Demand Pattern	Probability of Different Incident Types					
	No Incident	Shoulder Closure	One Lane Closure	Two Lane Closure	Three Lane Closure	Four Lane Closure
1	74.05%	18.14%	5.38%	1.35%	1.08%	0.00%
2	67.68%	22.45%	6.78%	1.72%	1.37%	0.00%
3	66.98%	22.92%	6.94%	1.76%	1.40%	0.00%
4	69.60%	21.16%	6.35%	1.61%	1.28%	0.00%
5	65.12%	24.17%	7.36%	1.87%	1.49%	0.00%
6	65.02%	24.24%	7.38%	1.87%	1.50%	0.00%
7	69.76%	21.05%	6.32%	1.60%	1.28%	0.00%
8	65.64%	23.82%	7.24%	1.84%	1.47%	0.00%
9	65.52%	23.90%	7.27%	1.84%	1.47%	0.00%
10	70.81%	20.34%	6.09%	1.54%	1.23%	0.00%
11	67.00%	22.91%	6.93%	1.76%	1.40%	0.00%
12	66.47%	23.26%	7.05%	1.79%	1.43%	0.00%

The analyst needs to enter FFS adjustment factor for different incident types for different number of lanes.

Number of Lanes (1 Direction)	No Incident	Shoulder Closure	One Lane Closure	Two Lane Closure	Three Lane Closure	Four Lane Closure
2	1.00	1.00	1.00			
3	1.00	1.00	1.00	1.00		
4	1.00	1.00	1.00	1.00	1.00	
5						
6						
7						
8						

The analyst needs to enter Capacity Adjustment Factors for different incident types for different number of lanes.

Number of Lanes (1 Direction)	No Incident	Shoulder Closure	One Lane Closure	Two Lane Closure	Three Lane Closure	Four Lane Closure
2	1.00	0.81	0.70			
3	1.00	0.83	0.74	0.51		
4	1.00	0.85	0.77	0.50	0.52	
5						
6						
7						
8						

Figure A.20. Option B section of the incident worksheet.

unique combination of demand, weather, and incident characteristics that is applied as matrices of DAFs, free-flow SAFs, and CAFs to the seed file data. As a general rule, the number of scenarios generated depends on a variety of factors, including the number of demand patterns selected, the diversity in weather activities, and the maximum number of lanes on any segment of the facility. Because incidents, weather, and demand are taken to be independent events, the number of total possible scenarios typically will be in the thousands.

To economize on run time, some very unlikely (low-probability) scenarios can be eliminated from consideration before running the core computational model. In this case, the user should specify a percentage threshold for filtering such scenarios. This process can be repeated by varying the threshold value and observing the trade-off between the resulting number of scenarios and the fraction of the distribution coverage (upper-right button in Figure A.21). Once a threshold is selected, clicking Step 5A: Generate/Update Scenarios will generate the scenarios, and the summary table will be populated with the summary information of the generated scenarios. Figure A.21 shows the schematic of the detailed scenarios worksheet in the FSG.

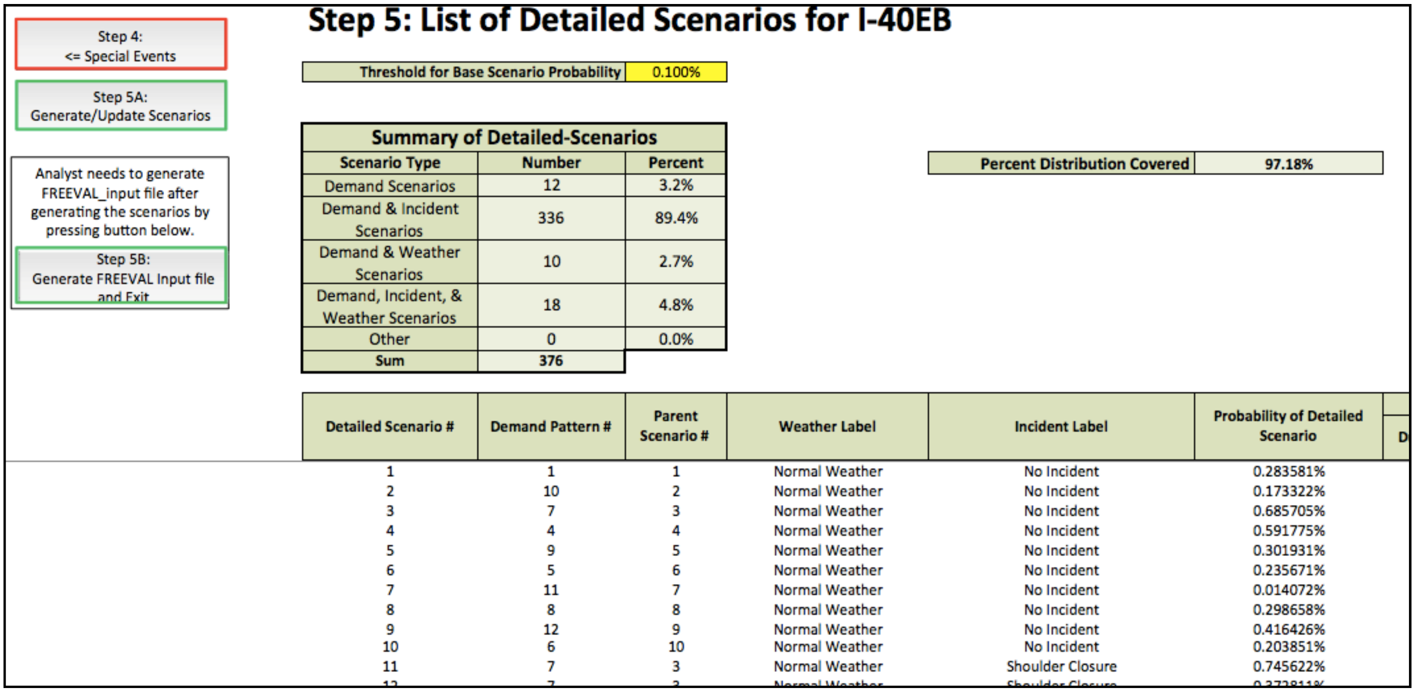


Figure A.21. Detailed Scenario worksheet in the FSG.

As a last step, the adjustment factor file for FREEVAL-RL is invoked by clicking Step 5B: Generate FREEVAL Input file and Exit. The FSG then generates the FREEVAL input (adjustment) file, exits the FSG, and returns the user to the FREEVAL-RL main menu.

At this point, the FSG will generate a file that contains tables with DAFs, SAFs, CAFs, and the number of lanes for each segment in each analysis time period. For a large facility (70 segments) with a long study period (6 h, i.e., 24 time periods), each of these tables will be in the form of a 70 × 24 matrix. A different set of tables is generated for each of what could be thousands of scenarios, which can result in a relatively large data file. Consequently, it may take a few minutes to complete this step.

It is important to note that no actual freeway facility analysis runs have been performed at this time. The FSG output file now serves as the input file for the FREEVAL-RL batch run performed in the next step.

The analyst should make any desired changes to the scenario file (e.g., modified demand, weather, incident inputs, or excluded scenarios) at this time, before running the much more time-consuming next step in FREEVAL-RL.

Step 4. Run Scenarios

When all scenarios have been generated using the FSG tool, its output is automatically transferred to FREEVAL-RL. A green check mark will appear beside the Scenario Management button, and the Run Scenarios button will be enabled (see Figure A.22). After clicking Run Scenarios, the tool will run the core computational engine in batch mode to execute all the scenarios generated by the FSG. Each scenario is analyzed, and its output is saved in a separate worksheet.

When all the scenarios are processed, the FREEVAL-RL file is automatically saved as a Detailed Output file, and a

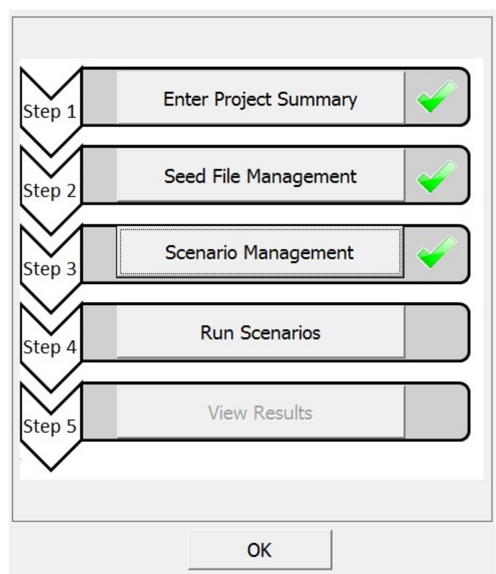


Figure A.22. Scenario Management step is finished and Run Scenarios step is active.

Summary Output file is saved, both in the same directory of the FREEVAL-RL. The Summary Output file consists of detailed information about each individual scenario and its respective output summary.

When the runs are completed, five files will be created in the FREEVAL-RL directory as follows (suppose the default file name entered in the first step is "I-40-Project"):

1. I-40-Project_FREEVAL-RL.xlsm is the seed file and main menu access.
2. I-40-Project_Freeway Scenario Generator.xlsm is the scenario generator file associated with the seed file scenarios. (See Item 1.)
3. FREEVAL_input.xlsm is a file for internal use only; it is read by FREEVAL-RL to execute all the scenarios. It contains all data generated by the FSG.
4. I-40-Project_ComprehensiveOutput.xlsm contains detailed output of the last scenario (only) in FREEVAL-RL in addition to facility detailed output for each scenario and analysis period. The one-page summary report is also part of this file, which is described in more detail below. This file may require a larger amount of space than other files.
5. I-40-Project_SummaryOutput.xlsm is the summary output file, which has only one worksheet with all run outputs in tabular format. Refer to the table titled Summary Output (Matrix) Description at the end of Appendix A for more information on the variables contained in this file.

Step 5. View Results

In the last step of the procedure, the user may view the summary output file by clicking View Summary Output or view a condensed one-page summary report by clicking View Summary Report (see Figure A.23).

A listing of the variables contained in the Summary Output is provided in the table at the end of Appendix A. A screenshot of a summary report is shown in Figure A.24, and a more detailed description of the various sections is available below.

In general, all statistics in the summary report are weighted by the VMT of traffic served in the respective scenarios. In other words, the estimates of travel time and other performance measures are weighted by the amount of traffic served in each scenario. A scenario with more demand will therefore weigh more heavily in the overall results than a low-demand scenario.

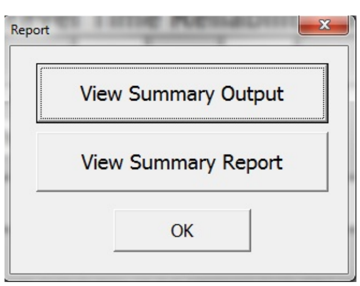


Figure A.23. View results buttons.

The motivation for weighing all results by the VMT is to approximate the performance as experienced by the traveler. For example, the VMT-weighted distribution of the travel time index (TTI) reflects the distribution as observed by drivers, with scenarios that affect more drivers (higher VMT) contributing more probability than low-VMT scenarios. These results are therefore different from the results that would be obtained from simply calculating an arithmetic average of the TTI across all scenarios.

The header of the output report is automatically populated with "Reliability Analysis Summary Report for" and then the name of the facility as entered by the user. The facility description block of the output report gives basic information about the length of the facility, the number of scenarios, and the number of scenarios with weather or incidents, or both. Figure A.25 shows the next block containing the overall reliability statistics for the facility.

The basic performance measures contained in Figure A.25 are as follows:

Mean TTI: The average, VMT-weighted TTI on the facility.

PTI: The planning time index for the facility, which is defined as the 95th percentile of the VMT-weighted cumulative TTI distribution. This measure is useful for estimating how much extra time travelers must budget to ensure an on-time arrival and for describing near-worst-case conditions on urban facilities.

80th percentile TTI: This facility performance measure is also VMT weighted. This measure has been found to be more sensitive to operational changes than the PTI, which makes it useful for comparison and prioritization purposes.

Misery index: This measure is defined as the average of the highest 5% of the TTI distribution divided by the free-flow facility travel time. This measure is useful as a descriptor of near-worst-case conditions on rural facilities.

Standard Deviation: This measure is the standard deviation of the VMT-weighted TTI distribution.

Semi-Standard Deviation: This measure is a one-sided standard deviation, with the reference point at free-flow speed instead of the mean. It provides the variability distance from free-flow conditions.

Percent of VMT at TTI > 1.33: This measure is a failure criterion that approximates the approximate number of trips at a speed 33% lower than the free-flow speed, which approximately coincides with the speed at capacity for freeways with a 70-mph free-flow speed.

Reliability Analysis Summary Report for Sample Facility

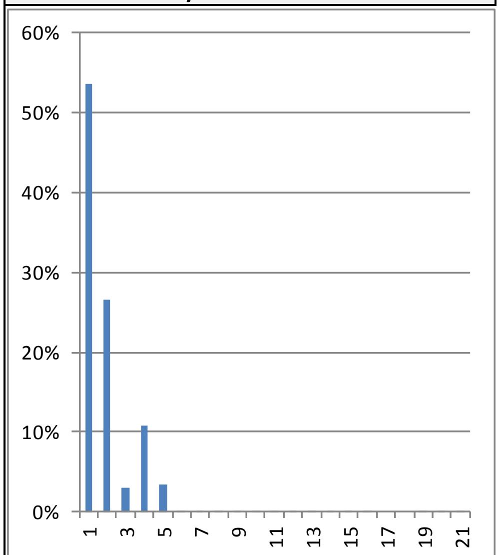
Facility Description

Number of Segments	4	Number of Weather Scenarios	80
Facility Length (miles)	1.0	Number of Incident Scenarios	192
Number of Total Scenarios	334	Numb. of Incident + Weather Scen.	54

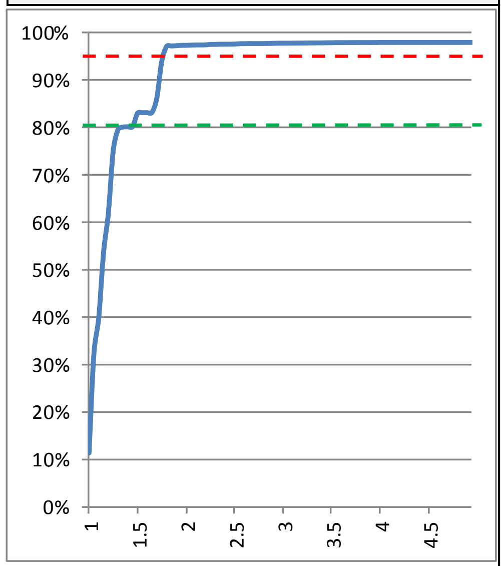
Facility Reliability Performance Measures

Mean TTI	1.25	Standard Deviation	5.10
PTI	1.77	Semi-Standard Deviation	5.58
80th percentile TTI	1.32	Percent of VMT at TTI > 1.33	51.73%
Misery index	3.30	Percent of VMT at TTI > 2	25.97%

Probability Distribution Function

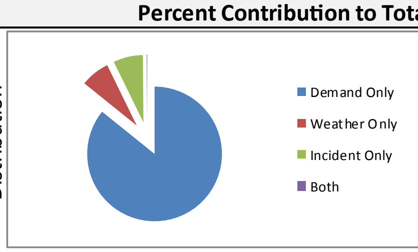


Cumulative Distribution Function

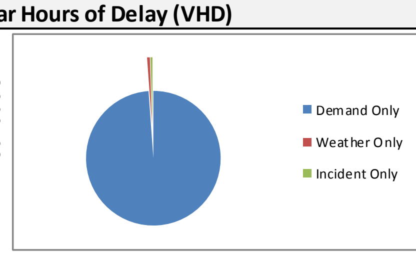


Percent Contribution to Total Vehicular Hours of Delay (VHD)

Overall VHD Distribution



Net Incremental VHD Distribution



Analysis Details for Reliability Reporting Period

Recurring Congestion (Demand Only)		**Non-Recurring Congestion Only**	
Maximum d/c Ratio	1.2	Maximum d/c Ratio	3.48
Maximum TTI	1.8	Maximum TTI	13.2
% Time with Queuing	2.9284%	% Time with Queues	0.0004%
% Time without Queues	97.07%	% Time without Queues	100.00%
Percent of VMT in Recurring-Only	2.6%	Percent of VMT in All Periods	97.4%
Total % Unserved VMT in RRP	100.00%	Total % Unserved VMT in RRP	96.28%

Figure A.24. One-page summary report.

Facility Reliability Performance Measures			
Mean TTI	1.25	Standard Deviation	5.10
PTI	1.77	Semi-Standard Deviation	5.58
80th percentile TTI	1.32	Percent of VMT at TTI > 1.33	51.73%
Misery index	3.30	Percent of VMT at TTI > 2	25.97%

Figure A.25. Facility reliability statistics.

Percent of VMT at TTI > 2: This failure criterion approximates the approximate number of trips at a travel time twice the free-flow travel time, which coincides with an average speed of half the free-flow speed.

In addition to the statistics shown in Figure A.25, a series of output graphs for the facility reliability analysis are available (see Figure A.26). These graphs show the probability density function and cumulative distribution function of the facility TTI, with both distributions being VMT weighted. The cumulative distribution function further highlights the 80th and 95th percentile TTIs.

Figure A.26 also shows the percentage distribution of delay by the various sources of congestion using two aggregation methods. The left chart shows the overall distribution of vehicle hours of delay (VHD) across all scenarios. The chart is a VMT-weighted breakdown of congestion sources for the total delay on the facility. It should be noted that the weather and incident VHD estimates in this case include some delay that would have occurred from demand impacts within the weather and incident scenarios.

The chart on the right is an alternative way of showing the VHD distribution that isolates the incremental delay. Conceptually, the VHD for each incident and weather scenario is

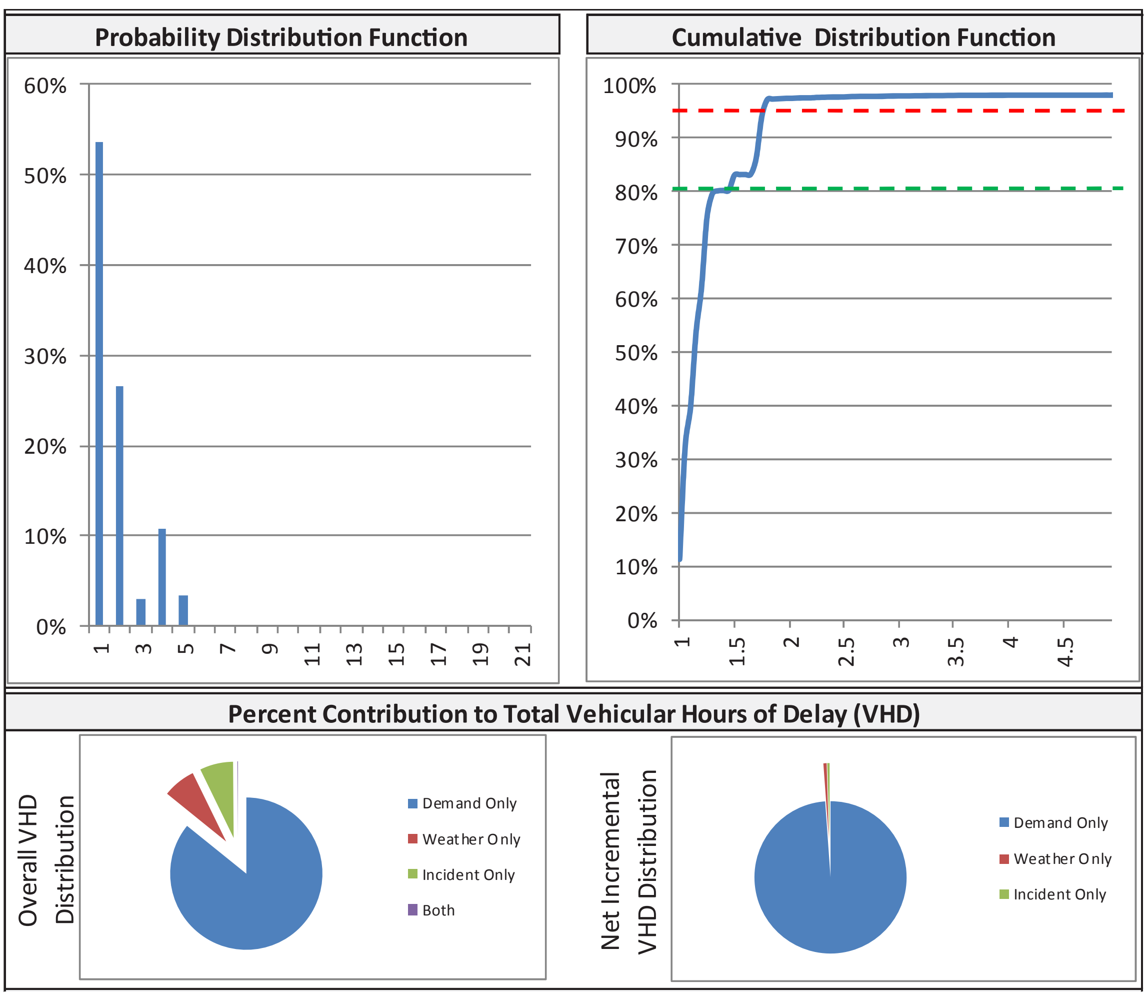


Figure A.26. Travel time distribution and output charts.

Analysis Details for Reliability Reporting Period			
Recurring Congestion (Demand Only)		**Non-Recurring Congestion Only**	
Maximum d/c Ratio	1.2	Maximum d/c Ratio	3.48
Maximum TTI	1.8	Maximum TTI	13.2
% Time with Queuing	2.9284%	% Time with Queues	0.0004%
% Time without Queues	97.07%	% Time without Queues	100.00%
Percent of VMT in Recurring-Only	2.6%	Percent of VMT in All Periods	97.4%
Total % Unserved VMT in RRP	100.00%	Total % Unserved VMT in RRP	96.28%

Figure A.27. Output details for recurring and nonrecurring congestion.

reduced by the amount of VHD that would have occurred from recurring sources of congestion (i.e., demand variability only).

A final part of the output includes additional statistics that separate recurring (demand-only) and nonrecurring (weather and incidents) scenarios, as shown in Figure A.27.

Figure A.27 shows the maximum demand-to-capacity (d/c) ratio and maximum TTI for each of the two groups of scenarios. It also shows the amount of time with and without queuing. Finally, it shows the percentage of VMT that is represented by the two groups and gives a sense of any unserved VMT in the RRP.

In addition to creating this standard output report, the user may decide to perform customized calculations, and a separate summary output file is provided for that purpose. The table at the end of Appendix A contains the listing and definitions of all variables included in that output.

MS Excel 2010 Security Options Quick Guide

Macros facilitate many tasks for MS Excel users. Many are created with Visual Basic for Applications and are written by software developers. However, some macros pose a potential security threat. A person with malicious intent can introduce a destructive macro in a document or file that can spread a virus on computers.

MS Excel does not enable macros automatically (i.e., by default). In the 2010 version, an option for enabling macros is provided in the welcome notification. Figure A.28 shows the button that enables macros in the MS Excel 2010 version.

Please note that the FREEVAL-RL computational engine uses multiple macros embedded behind the user interface. The user must enable macros in order to run the computational engine.

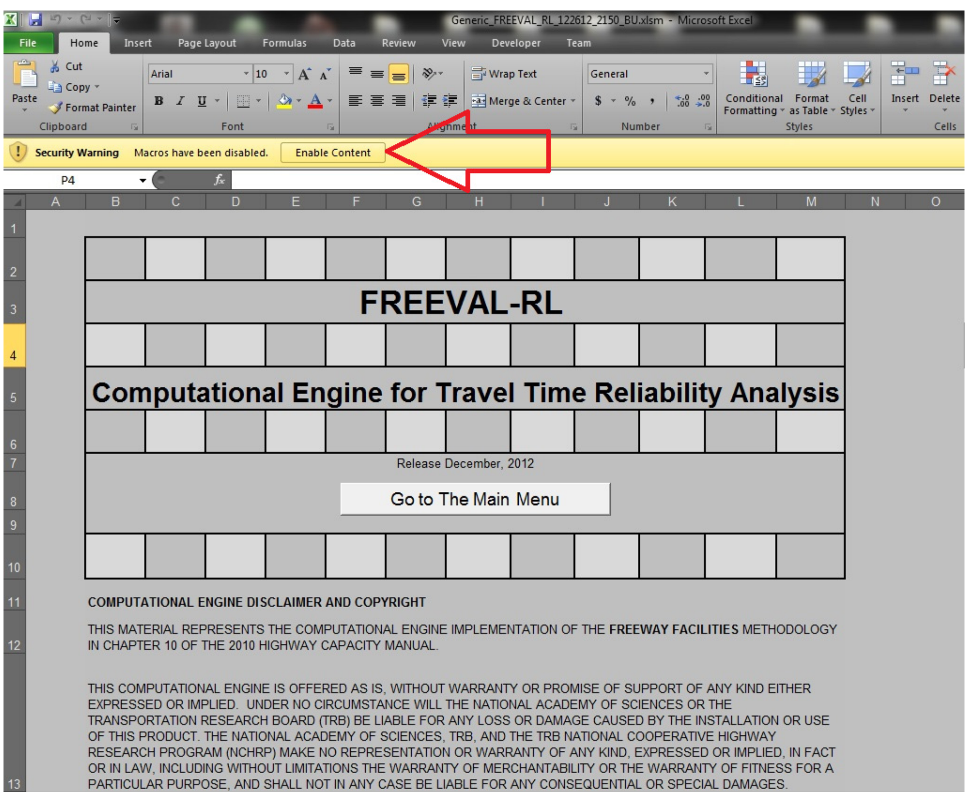


Figure A.28. Enabling macros before running FREEVAL-RL.

Summary Output (Matrix) Description

Entry	Description
Scenario Number	Scenario Number
Parent Scenario Number	A scenario with *normal weather, no incident* with an identical demand pattern as the current scenario is called the Parent Scenario. For weather and incident scenarios, the *weather only* and *incident only* scenarios are called subparent scenarios. Each reliability scenario therefore has (only) one parent scenario. This attribute is useful when estimating additional delay due to weather and/or incidents relative to the demand-only parent scenario.
Analysis Period	Analysis period No. (varies from 1 to number of analysis periods in study period)
Probability	Probability of a scenario (from FSG)
Demand Adjustment Factor	A multiplicative factor of demand relative to the base scenario
Weather Type	Weather condition description in the scenario
Weather Event Start Time	Start time of the weather event (either start or middle of the study period)
Weather Event Duration (min)	Duration of the weather event in minutes
Weather Event CAF	Capacity adjustment factor due to the weather event
Weather Event SAF	Speed adjustment factor due to the weather event
Incident?	A Boolean value indicating the presence of an incident in the study period: 0 for no incident, 1 for incident
Incident Start Time	Start time of the incident (start or middle)
Incident Duration (min)	Duration of the incident in minutes
Incident Segment Number	Segment number where the incident occurs
Segment Number of Lanes	Total number of lanes on the incident segment
Number of Closed Lanes	Total number of lanes closed due to the incident
Per Open Lane Incident CAF	Capacity adjustment factor applied to each of the open lanes as due to the incident
Incident SAF	Speed adjustment factor of the incident (defaulted at 1.0)
TTI	Facility travel time index in the analysis period
Max d/c Ratio	Maximum demand-to-capacity ratio for all the segments in the analysis period
Queue Length (ft)	Queue length at the end of the analysis period
Total Denied Entry Queue Length (ft)	Queue length of vehicles unable to enter the facility at the first segment
Total On-Ramp Queue Length	Queue length of vehicles on on-ramps
Average Travel Time per Vehicle (min)	Average travel time experienced by each vehicle traveling the facility in the analysis period
Free-flow Travel Time (min)	Facility travel time experienced by each vehicle if it traveled at free-flow speed
Freeway Mainline Delay (min)	Delay experienced per vehicle. Calculated by subtracting free-flow travel time from average travel time per vehicle.
System Delay—Includes On-Ramp (min)	Total delay of the analysis period is the summation of mainline delay and all on-ramp delays.
VMTD Demand	Vehicle miles traveled as if all demand had been served in the analysis period
VMTV Volume	Vehicle miles traveled of the vehicles actually served during the analysis period
VHT travel/interval (h)	Vehicle hours traveled by all served vehicles during the analysis period
VHD delay/interval (h)	Vehicle hours of delay experienced by all served vehicles during the analysis period
Space mean speed = VMTV/VHT (mph)	Space mean speed at the analysis period calculated by dividing served vehicles miles traveled by total vehicles hours of travel
Facility Average Density (pc/mi/lane)	Average density on the facility in passenger cars per mile per lane
Density-Based Facility LOS	Facility level of service based on the facility average density
Demand-Based Facility LOS	Facility level of service based on demand

Reference

Highway Capacity Manual 2010. Transportation Research Board of the National Academies, Washington, D.C., 2010.

APPENDIX B

STREETVAL User's Guide

This appendix provides guidance for the use of the Urban Streets Reliability Engine (USRE), referred to as STREETVAL in the proposed *Highway Capacity Manual* (HCM) reliability chapters. USRE is a software tool that supports the evaluation of a facility's reliability of service in terms of its operational performance over an extended time period. The software is distributed as a Microsoft (MS) Excel workbook using the Visual Basic for Applications (VBA) programming language.

This user's guide consists of six sections:

- An introduction to the tool and description of its correct use;
- The evaluation process in terms of the analysis steps and the data needed for a typical facility evaluation;
- Supplemental guidance for assigning reported crashes to intersections and streets;
- Computational engine data entry;
- Software setup and file description; and
- Software installation.

Introduction

Overview

The USRE (hereafter referred to as the "tool") was developed to predict the operational performance of a facility for each of many small time periods that collectively represent traffic conditions during several consecutive months. The predictions are used to describe the distribution of various performance measures, notably travel time, over the duration of interest. The distribution provides an indication of the degree to which the facility provides reliable service.

Two methodologies are implemented in the tool. The first one, reliability methodology, is used to estimate traffic, signal, road, and weather conditions for each of the small time periods. The estimates are based on historic changes in these conditions over time, with the recognition that there is also a random component to these changes in terms of when and where they occur, as well as in their magnitude or duration.

The second methodology is that documented in Chapters 16, 17, and 18 of the *2010 Highway Capacity Manual* (HCM2010) (Transportation Research Board of the National Academies 2010). For this reason, it is referred to as the HCM methodology. It is used to predict facility travel time and other performance measures for each small time period based on its estimated traffic signal, road, and weather conditions.

This guide was developed as a companion document for the proposed HCM reliability chapters about the recommended methodology for incorporating travel time reliability into the HCM. These chapters provide the information needed to fully understand and apply the reliability methodology. Specifically, they identify the required input data, describe the methodology, provide default values, outline some typical applications, and provide a detailed example problem. Analysts are encouraged to read the proposed HCM reliability chapters before using the tool.

Evaluation Scope

The tool is intended to be used to quantify the reliability of service provided by an urban street facility in terms of its operational performance over an extended period of time. The evaluation focuses on the performance of automobile and truck traffic and does not directly address the performance of other urban street travel modes. The urban street facility can be either an arterial or collector street. Each segment on the facility is bounded by a signalized intersection.

The reliability methodology can be used to evaluate the following sources of unreliable travel time on urban street facilities:

- Traffic incidents;
- Work zones;
- Demand fluctuations;

- Special events;
- Traffic control devices;
- Weather; and
- Inadequate base capacity.

These sources can result in the formation of oversaturated operation for extended time periods.

Traffic demand fluctuations are represented in the reliability methodology in terms of systematic demand volume variation by hour of day, day of week, and month of year. Fluctuations due to diversion are not addressed by the methodology.

Traffic control devices are represented in the HCM methodology in terms of the influence of speed limit and traffic signal operation on facility travel time. This sensitivity can be used to evaluate the quality of the signal timing plan and the relative benefits of pretimed or coordinated-actuated operation. The effect of traffic-responsive or -adaptive signal operation is not addressed by the methodology.

Inadequate base capacity is represented in the reliability methodology as the potential for a signalized intersection to act as a bottleneck to traffic flow along the facility. This result may be due to a lane being dropped at the intersection, insufficient numbers of intersection approach lanes, or misallocation of cycle time. The effect of a midsegment lane drop or weaving section is not addressed by the methodology.

Limitations

The reliability methodology does not address some events (or conditions) that influence urban street operation. The inability to quantify the influence of an event or condition on traffic operation represents a limitation of the methodology. This subsection identifies the known limitations of the reliability methodology. If one or more of these limitations are believed to have an important influence on the performance of a specific facility, then the analyst should consider using alternative evaluation methods or tools.

The reliability methodology does not directly account for the effect of the following conditions on facility operation:

- Truck pickup and delivery;
- Signal malfunction;
- Railroad crossing;
- Railroad preemption;
- Signal plan transition; or
- Fog, dust storms, smoke, or high winds.

Lane or shoulder blockage due to truck pickup and delivery activities in downtown urban areas can also be considered incidentlike in terms of the randomness of their occurrence and the temporal extent of the event. The dwell time for these activities can range from 10 to 20 min.

A signal malfunction occurs when one or more elements of the signal system are not operating in the intended manner. These elements include vehicle detectors, signal heads, and controller hardware. A failure of one or more of these elements typically results in poor facility operation.

A railroad crossing the facility at a midsegment location effectively blocks traffic flow while the train is present. Train crossing time can be lengthy (i.e., typically 5 to 10 min) and can cause considerable congestion that can extend for one or more analysis periods.

Railroad preemption is used when a train crosses a leg of a signalized intersection. The signal operation is initially disrupted to safely clear the tracks. It then dwells in a specified phase sequence while the train is present. Signal coordination may be disrupted for several cycles following train clearance.

When a new timing plan is invoked, the controller transitions from the previous plan to the new plan. The transition period can last several cycles, during which traffic progression is significantly disrupted.

Some weather conditions that restrict driver visibility or degrade vehicle stability are not addressed by the methodology. These conditions include fog, dust storms, smoke, and high winds. They tend to be localized to specific areas of the country and are relatively rare in occurrence.

The reliability methodology uses the HCM methodology to quantify facility performance during each scenario. For this reason, the reliability methodology shares the limitations of the HCM methodology. These limitations are described in Section 1 of Chapters 16, 17, and 18 of the HCM2010.

Software Limits

The tool can accommodate data for eight segments and nine signalized intersections. If a given project exceeds one or more of these limits, the analyst will need to subdivide the project into two or more sections such that each section does not exceed the limits.

The tool can address the occurrence of up to seven work zones, special events, or both. The total number of work zones and special events cannot exceed seven.

Terminology

This section defines many of the terms used in this document. Those terms that are not listed here are defined in Chapter 9 of the HCM2010.

Analysis Period

The analysis period is the time interval evaluated by a single application of the HCM methodology.

HCM Data Set

An HCM data set comprises the input data needed to evaluate an urban street facility using the HCM methodology. These data are also needed for the reliability methodology. They are described in Chapter 17 of the HCM2010 and are referred to here as an "HCM data set." One data set describes the geometry and signal timing conditions for the intersections and segments on the facility during one representative study period. The demand volumes recorded in a data set describe conditions for a specified hour of the day and date of the year. Alternatively, they can be stated to represent the average day of the year if they are derived from an annual average daily traffic (AADT) volume.

For many reliability evaluations, there will be two or more HCM data sets. One HCM data set describes base conditions. This base data set is required for a reliability evaluation. The base conditions describe demand volume, geometry, and signal timing conditions when work zones and special events are not present. The demand volume for base conditions should be representative of good weather (i.e., it should not represent traffic movement counts during rain or snow storms).

Additional HCM data sets are used, as needed, to describe conditions when a specific work zone is present or when a special event occurs. These optional data sets are called alternative data sets. One alternative data set is used for each time period during the reliability reporting period (RRP) when a specific work zone is present, a specific special event occurs, or a unique combination of these occurs during the study period.

Scenario

A scenario is a unique combination of traffic demand, geometry, and traffic control conditions. It can represent one or more analysis periods, provided that all periods have the same unique combination of demand, capacity, geometry, and traffic control.

Study Period

The study period is a time interval (within a day) that is represented by the performance evaluation. It consists of one or more consecutive analysis periods.

Reliability Reporting Period

The RRP describes the specific days over which reliability is to be computed, such as all nonholiday weekdays in a year.

Special Event

Special events are short-term events (e.g., major sporting events, concerts, and festivals) that produce intense traffic demands on a facility for limited periods of time and that may be addressed by temporary changes in the facility's geometry, traffic control characteristics, or both.

Getting Started

This section introduces the tool and describes its correct use. It consists of the following six subsections:

- Enabling macros: guidance for setting spreadsheet security to enable macros;
- Navigation: guidance for selecting and using the worksheets;
- Entering data: guidance for entering data in a worksheet;
- Reviewing results: guidance for reviewing, saving, and printing results;
- Modifying calibration factors and default values: guidance for calibrating the tool to local conditions; and
- File management: guidance for saving and deleting files created by the tool.

Enabling Macros

The tool contains VBA computer code, referred to as macro code in MS Excel, to automate the calculations. It must be enabled when first loading the tool into Excel. The technique for enabling macros varies depending on the version of Excel being used.

Enabling Macros in Microsoft Excel 2003

The following instruction sequence enables macros for Excel 2003. Open the Excel software. From the main screen, click Tools, and then Options. In the Options panel, click Security, and then click Macro Security. In the Security panel, click Security Level, and then click the radio button adjacent to Medium (the button will show a black circle). Finally, click OK to exit the Security Level panel and click OK to exit the Options panel. This setting should only need to be set once. It will remain effective until this process is repeated and a new security level is selected.

Every time the tool is opened in Excel, the pop-up box shown to the right will be displayed. The analyst should click Enable Macros. The tool will finish loading and will function as intended.

Enabling Macros in MS Excel 2007 or 2010

The following instruction sequence enables macros for Excel 2007 or 2010. Open the Excel software to the main screen. For Excel 2007, click Office, and a panel will be displayed. In this panel, click Excel Options to bring up the Excel Options panel. For Excel 2010, click File, then click Options to bring up the Excel Options panel.

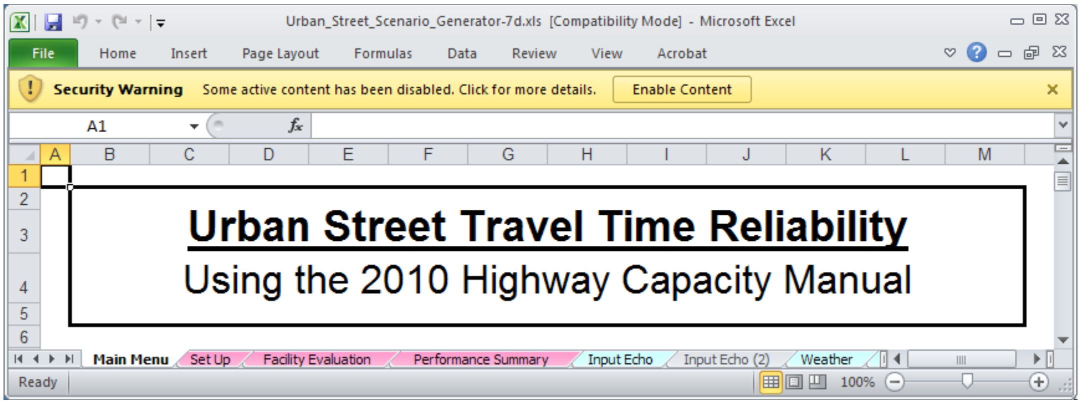


Figure B.1. Urban street scenario generator.

For Excel 2007 or Excel 2010, while in the Excel Options panel, click Trust Center, and then click Trust Center Settings to bring up the Trust Center panel. In this panel, click Macro Settings and then click the radio button adjacent to Disable all macros with notification (the button will show a black circle). Finally, click OK to exit the Trust Center panel, and click OK to exit the Excel Options panel. This setting should only need to be set once. It will remain effective until this process is repeated and a new security level is selected.

Every time the tool is opened in Excel, a security warning is displayed. It is shown near the top in the following graphic for Excel 2010. A similar message is shown in Excel 2007. In Excel 2007, the analyst should click the Options button, click Enable this content, and then click OK. In Excel 2010, the analyst should click Enable Content (see Figure B.1).

Navigation

The tool contains 12 worksheets. To navigate among worksheets, click the worksheet tabs at the bottom of the workbook window. The worksheets are identified in the following list:

- Main Menu: includes a foreword, instructions, acknowledgment, and disclaimer;
- Set Up: basic input data, name of each detailed input data file, start scenario generation;
- Facility Evaluation: input data describing evaluation interval, start scenario evaluation;
- Performance Summary: input data describing measure of interest, start performance summary;
- Input Echo: listing of the data in a detailed input data file;
- Weather: detailed listing of the predicted weather characteristics for each analysis period;
- Demand: detailed listing of the demand variation factors applicable to each analysis period;
- Incident: detailed listing of the predicted incident characteristics for each analysis period;
- Calib-Weather: calibration factors and default data for weather;
- Calib-Demand: calibration factors and default data for demand volume;
- Calib-Incident: calibration factors and default data for incidents; and
- Analysis: statistical analysis of results from multiple runs.

The Set Up, Facility Evaluation, and Performance Summary worksheets will be used for every evaluation in the order listed. Optionally, the Weather, Demand, or Incident worksheets could be used to examine detailed results for one or more specific analysis periods. The three calibration worksheets should be used to adjust the calibration factors or replace the default data (or both) to reflect local conditions.

Entering Data

Data are entered in the Set Up, Facility Evaluation, and Performance Summary worksheets. The basic data entry considerations are the same for each worksheet. A sample portion of the Set Up worksheet is shown in Figure B.2 to illustrate these considerations. The guidance offered in this section applies to all input worksheets.

In all input worksheets, the cells with a light-blue background are for user input. The white cells and gray cells are not for input, so they are locked to prevent inadvertent changes to cell content.

The red triangles in the upper-right corner of some cells are linked to supplemental information balloons. Red triangles are shown for five cells in Figure B.2. By positioning the mouse pointer over a red triangle, a balloon will appear. In it will be information relevant to the adjacent cell that will typically explain more precisely what input data are needed.

A drop-down list is provided for some cells with a light-blue background. When one of these cells is selected, a gray button will appear on the right side of the cell. Position the

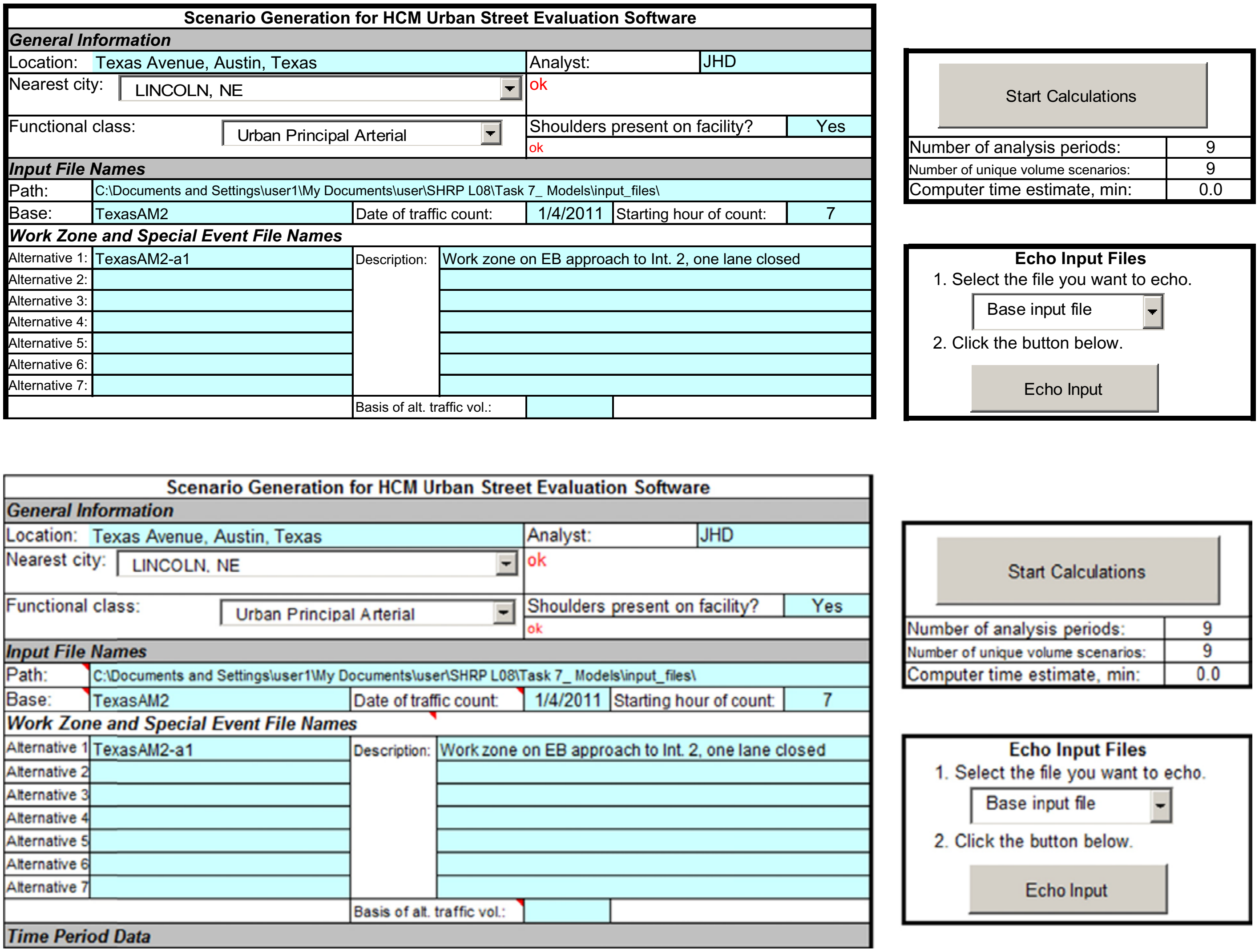


Figure B.2. Set Up worksheet.

mouse pointer over the button and click the left mouse button. After clicking this button, a list of input choices will appear. Use the mouse pointer to select the desired choice, and then click the left mouse button.

The section of Figure B.2 titled General Information shows two drop-down windows. On the right side of each list there is a gray button. Position the mouse pointer over the button and click the left mouse button. After clicking this button, a list of input choices will appear. Use the mouse pointer to select the desired choice, and then click the left mouse button.

On the right side of Figure B.2 are two gray buttons. One button is labeled Start Calculations. Clicking it will initiate the scenario generation process. A similar process-starting button is provided in the Facility Evaluation worksheet and the Performance Summary worksheet. After clicking a process-starting button, progress through the calculation sequence can be monitored by viewing the counter in the lower left corner of the Excel software (i.e., the status bar). "Ready" is shown before the button is clicked. A numeric counter is shown after the button is clicked.

The second gray button, labeled Echo Input, will list the data in a selected input file. The file of interest is identified using the drop-down list just above the Echo Input button. The listing will be displayed in the Input Echo worksheet.

Reviewing Results

This subsection provides guidance for reviewing and saving the evaluation results. The summary results are shown in the Performance Summary worksheet. In addition to summary results, this worksheet also lists the performance measure for every analysis period (i.e., scenario) in the RRP. Detailed listings of the predicted weather characteristics, demand variation factors, and incident characteristics for each analysis period are provided in the Weather, Demand, and Incident worksheets, respectively.

The evaluation results can be saved by saving the entire workbook. The File, Save As menu sequence should be selected, and a new file name entered when prompted (i.e., avoid overwriting the original workbook). Alternatively, the results from the Performance Summary worksheet can be copied and pasted into another worksheet for supplemental data summary, aggregation, or plotting.

Modifying Calibration Factors and Default Values

The reliability methodology has been developed using data from facilities in several urban areas. In many instances, these data are represented in the tool as calibration factors or default values. Adjusting these factors and values to local conditions will account for any differences between the facilities used for development and those being evaluated and will ensure that the evaluation results are meaningful and accurate for the jurisdiction.

The calibration factors and default values are located in the Calib-Weather, Calib-Demand, and Calib-Incident worksheets (i.e., the calibration worksheets). Most of the variables identified in these worksheets are considered to be default values because they have been found to notably influence the evaluation results, and they are typically available from archival databases or simple field measurements. A description of the default values is provided in the proposed HCM reliability chapters.

A few of the variables in the calibration worksheets are considered to be calibration factors either because they have a relatively nominal effect on the evaluation results or they are not readily available or easily measured in the field. Background information about the calibration factors is provided in Appendix H.

File Management

One base data set and, optionally, up to seven alternative data sets are used by the tool to generate the conditions present for each analysis period (i.e., scenario). The base and alternative data sets are created by the analyst and saved in a folder. The path to this folder is input by the analyst in the Set Up worksheet. Specifically, it is input in the first row below the section labeled Input File Names, as shown in Figure B.2. The file name for the base file is input in the next row down. The file name for each of the alternative data sets is input in the rows below the subsection labeled Work Zone and Special Event File Names, as shown in Figure B.2. Guidance for setting up the file structure is provided in the section titled "Software Setup and File Description."

After the Start Calculations button is clicked, the tool generates one new data set for each analysis period in the RRP. Each data set is saved in a file using a file name with a .txt extension. The file name includes the text of the file name associated with the base (or alternative) data set used to create the new data set. In addition, the file name is expanded to include the date and starting time of the corresponding analysis period. Each file generated in this manner is saved in the same folder, as is the base data set. The typical reliability evaluation will generate several thousand of these files.

After the Evaluate Scenarios button is clicked, the tool evaluates each analysis period using the HCM methodology. The results from each evaluation are saved in a file using the same file name as the input value but with the extension .out. Thus, there is one .out file for each .txt file. Each file generated in this manner is saved in the same folder, as is the base data set. The typical reliability evaluation will generate several thousand of these files. The file name convention and format of these files is described in "Software Setup and File Description" on p. 164.

The analysis period data set files and the associated results files should be deleted when the evaluation is complete. Alternatively, they can be archived using a file compression utility.

Evaluation Process

This section describes the activities undertaken during the evaluation of an urban street facility. The first subsection describes the sequence of evaluation activities in the order they are conducted; these activities are outlined as a series of analysis steps. The second subsection describes the data input requirements for the reliability methodology. The third subsection describes the data entry process using the tool, and the fourth subsection describes the evaluation results.

The evaluation requires the creation of a base data set and, optionally, one or more alternative data sets. The Urban Streets Computational Engine (USCE) is used for this purpose. The USCE is a software tool that implements the HCM methodology. The software is packaged as a MS Excel workbook using VBA programming language. Guidance for using the USCE to create the base data set is provided in the section titled "Urban Streets Computational Engine Data Entry."

The predicted operational performance is summarized as the last step of the evaluation process. This performance is described in terms of the distribution of a selected performance measure, such as facility travel time. Improvement strategies can be devised and then evaluated through repetition of this process.

Analysis Steps

The steps involved in a safety evaluation using the tool are considered to be the routine steps that are used each time a safety evaluation is undertaken. These steps are identified as follows:

1. Define purpose and scope;
2. Divide facility into individual segments;
3. Acquire input data; and
4. Initiate calculations and review results.

Detailed information about each step is provided in the following subsections.

Step 1. Define Purpose and Scope

The purpose and scope of the evaluation are defined in this step. The purpose defines the nature and extent of the operational problems on the facility and the anticipated use of the information obtained from the evaluation (e.g., quantify problem, diagnose main causes, devise strategies).

The scope of the evaluation is used to define the spatial and temporal extent of the evaluation. The spatial extent is characterized by the project limits, which define the physical extent of the facility being evaluated (i.e., the number of consecutive segments). The temporal extent of the evaluation is defined by the duration of the analysis period, the hours of the day spanned by the study period, and the days of the year spanned by the RRP. The RRP is also defined by the days of the week to be considered in the evaluation.

Step 2. Divide Facility into Individual Segments

Using the project limits identified in Step 1, the facility is divided into segments, with each segment being bounded by a signalized intersection. Segments are internal to the facility and have signalized boundary intersections. The evaluation considers both directions of travel on a segment (when the segment serves two-way traffic flow).

Step 3. Acquire Input Data

Input data are acquired during this step. Required input data include those data normally needed to apply the methodology in Chapter 17 (Urban Street Segments) of the HCM2010. These data are needed to create the base data set.

If work zones or special events occur during the RRP, then additional data are needed. Specifically, one alternative data set is created for each work zone or special event. The analyst must specify any changes to base conditions (e.g., demand, traffic control, available lanes) associated with a work zone or special event, along with a schedule for when the alternative data set is in effect. For example, if a work zone exists during a given month, then an alternative data set is used to describe average conditions for the analysis period during that month. Work zones that exist at the same time as a special event must be described using one alternative data set.

In addition to the HCM data sets, the reliability methodology requires some input data to describe the reliability evaluation. These data are needed to define the time period (i.e., temporal scope) of the evaluation, characterize the project location, and describe the crash history of each street and intersection along the facility.

The specific data elements needed are described in the section titled "Input Data Requirements," and the means by which they are entered into the tool are described in "Data Entry."

Step 4. Initiate Calculations and Review Results

The calculations proceed in sequence through the scenario generation, facility evaluation, and performance summary stages of the evaluation process. The scenarios are generated first by clicking the Start Calculations button (shown in Figure B.2) in the Set Up worksheet.

Once the scenarios have been generated, the analyst moves to the Facility Evaluation worksheet and clicks Evaluate Scenarios, which initiates the evaluation of each scenario.

Finally, the analyst moves to the Performance Summary worksheet and clicks Summarize Results. This action initiates the process of gathering the selected performance measure data from the results files and summarizing them using selected distribution statistics.

Additional information about the Set Up, Facility Evaluation, and Performance Summary worksheets is provided below under "Data Entry."

When the three stages are complete, the analyst can examine the performance measure summary statistics to evaluate the overall operational performance of the facility. If more insight is needed about specific time periods, the predicted performance measure for each analysis period is available for examination. Details about the predicted weather, demand volume, or incidents during specific analysis periods can be examined in the Weather, Demand, or Incident worksheets, respectively. Additional information about the evaluations results is provided below under "Results Review and Interpretation."

Input Data Requirements

Two types of required input data are needed for the tool. One type of data is that normally needed to apply the urban street segments methodology in Chapter 17 of the HCM2010. These data, which are described in the HCM, are needed to create the base data set and, optionally, the alternative data sets.

Table B.1. Input Data

Category	Variable
General	Nearest city Functional class
Traffic counts	Date and time of traffic count for base data set Date and time of traffic count for alternative data set Peak hour factor
Geometry	Presence of shoulders
Time period	Analysis period duration Study period Reliability reporting period Alternative data set operating period
Crash	Segment crash frequency Intersection crash frequency Crash frequency adjustment factors

The second type of input data is that needed for the reliability evaluation. These data are listed in Table B.1 and are the focus of discussion in this section.

Nearest City

Of interest to the reliability evaluation are the weather statistics identified in the following list. The methodology uses these statistics when they are averaged by month of year for a recent 10-year period.

- Total normal precipitation (in.);
- Total normal snowfall (in.);
- Number of days with precipitation of 0.01 in. or more;
- Normal daily mean temperature (°F); and
- Precipitation rate (in./h).

The nearest city input is used to identify the typical weather conditions for the subject facility. Default values are available in the tool for 284 U.S. cities and territories. The first four statistics listed are published by the National Climatic Data Center (2011a), which also publishes the average precipitation rate for these locations in the Rainfall Frequency Atlas (National Climatic Data Center 2011b). The National Climatic Data Center or other weather data sources can be consulted to obtain the necessary weather statistics for cities for which default values are desired but not available in the tool.

Precipitation statistics include both rainfall and snowfall, where snowfall is measured by its liquid equivalent.

Functional Class

The functional class of the subject facility is used to estimate the traffic volume during each of the various scenarios that comprise the RRP. Specifically, it is used to determine the appropriate traffic volume adjustment factors for each scenario. The functional classes that are considered are identified in the following list:

- Urban expressway;
- Urban principal arterial street; and
- Urban minor arterial street.

An urban principal arterial street emphasizes mobility over access. It serves intra-area travel, such as that between a central business district and outlying residential areas, or that between a freeway and an important activity center. It is typically used for relatively long trips within the urban area, or through trips that are entering, leaving, or passing through the city. An urban minor arterial street provides a balance between mobility and access. It interconnects with and augments the urban principal arterial street system. It is typically used for trips of moderate length within relatively small geographic areas (American Association of State Highway and Transportation Officials 2011).

Default month-of-year, hour-of-day, and day-of-week adjustment values are provided for each functional class. These values are described in the proposed HCM reliability chapters.

Date and Time of Traffic Counts

The date and time of the traffic count represented in an HCM data set are used as a basis for estimating the traffic demand volume during each of the various analysis periods that comprise the RRP. Specifically, the date and time of the count are used to determine the hour-of-day, day-of-week, and month-of-year factors that are then used to convert the volumes in the base data set into average-day-of-year volumes. A similar adjustment is made to the volumes in the alternative data sets.

If the traffic demand volumes provided in the base data set (and the alternative data sets) are computed using planning procedures, then they are assumed to represent an average day volume. In this situation, a date does not need to be provided by the analyst. However, the time of day for which the estimated volumes apply is still needed.

Peak Hour Factor

If a 15-min analysis period is used, the analyst has the option of adding a random element to estimated volume for each movement and analysis period. Including this random element provides a more realistic estimate of performance measure variability. If this option is selected, then the analyst is asked to provide the peak hour factor for each intersection. This factor is then used to randomly adjust the turn-movement volumes at each intersection. The algorithm used for this adjustment was developed to ensure that the resulting volume variation among analysis periods in a common hour is consistent with that implied by the peak hour factor.

Presence of Shoulders

The presence of outside (i.e., right-side) shoulders is used to predict incident location. The default distribution of incident lane location is based on facilities with outside shoulders. This distribution is modified when shoulders are not present on the subject facility. For a shoulder to be considered present, it must be sufficiently wide to store a disabled vehicle without blocking traffic flow in the adjacent traffic lane.

If on-street parking is allowed, the analyst will need to determine whether its occupancy during the study period is sufficient to preclude its use as a refuge for disabled vehicles. It is judged that the proportion of on-street parking occupied would need to be less than 30% to provide reasonable assurance that there will be opportunity to move a disabled vehicle from the through lanes to an open stall.

Analysis Period Duration

The analysis period is the time interval considered for the performance evaluation. Its duration is in the range of 15 min to 1 h, with longer durations in this range sometimes used for planning analyses. A shorter duration in this range is typically used for operational analyses. Additional guidance for determining the analysis period duration is provided in Chapter 16 of the HCM2010.

A shorter analysis period duration is desirable for reliability evaluations because it reduces the minimum event duration threshold and thereby increases the number of incidents and weather events that are recognized. In this regard, the structure of the reliability methodology is such that events that are shorter than one-half of the analysis period duration are ignored (i.e., they will not be recognized in the scenario generation process). Thus, the use of a shorter analysis period duration will minimize the number of events that are ignored.

Study Period

The study period is the time interval (within a day) that is represented by the performance evaluation. It consists of one or more consecutive analysis periods. A typical study period is 1.0 to 6.0 h in duration and is stated to represent specific times of day and days of the week (e.g., weekdays from 4:00 to 6:00 p.m.). If congestion occurs during the study period, then at least the first analysis period should be uncongested. The maximum study period duration is 24 h.

The geometric design elements and traffic control features of the facility must be unchanged during this period. Thus, the intersection lane assignments and signal timing plan should be the same during the study period. If the facility has two or more time-of-day signal timing plans, then a separate study period should be established for each plan period. Similarly, if the directional distribution of traffic volume changes significantly during the day, then separate study periods should be established for each time period when the directional distribution is relatively constant.

Reliability Reporting Period

The RRP represents the specific days over which reliability is to be computed. A typical reporting period for a reliability evaluation is 6 to 12 months. It is specified by start and end dates, as well as the days of week being considered. The RRP is used with the study period to fully describe the temporal representation of the performance measure (e.g., average travel time during weekday periods from 4:00 to 6:00 p.m. for the current year).

Alternative Data Set Operating Period

One or more alternative data sets are used to describe conditions when a specific work zone is present or when a special event occurs. The operating period for each alternative data set is specified by its start and end dates.

Segment Crash Frequency

The segment crash frequency is used to predict incident occurrence on each of the segments that comprise the facility. The crash frequency that is input represents an estimate of the expected crash frequency for the segment when no work zones are present or special events occur. The estimate should include all severity levels, including property-damage-only crashes. It is provided in units of crashes per year, regardless of the duration of the RRP.

The segment crash frequency does not include crashes that occur at the intersection or crashes that occur on the intersection legs and are described in the crash report as "intersection related." The assignment of crashes to segments is described below under "Urban Streets Computational Engine Data Entry." The expected crash frequency can be computed using the predictive method in Chapter 12 of the *Highway Safety Manual* (American Association of State Highway and Transportation Officials 2010). If this method cannot be used, then a 3-year crash history for the subject segment can be used to estimate its expected crash frequency.

Crashes that occur when work zones and special events are present should be removed from the crash data. In this situation, the expected crash frequency is computed as the count of crashes during times when work zones and special events are not present divided by the time period when work zones and special events are not present. Thus, if there were 15 crashes reported during a recent 3-year period and five of these crashes occurred during a 6-month period when a work zone was present, then the expected crash frequency is estimated as 4.0 crashes per year ($[15 - 5] / [3 - 0.5]$).

Intersection Crash Frequency

The intersection crash frequency is used to predict incident occurrence at each of the intersections within the limits of the facility. The crash frequency that is input represents an estimate of the expected crash frequency for the intersection when no work zones are present or special events occur. The estimate should include all severity levels, including property-damage-only crashes. It is provided in units of crashes per year, regardless of the duration of the RRP. Guidance for obtaining this input data is provided in the preceding subsection, "Segment Crash Frequency."

Crash Frequency Adjustment Factor

The crash frequency adjustment factor is used to estimate the expected crash frequency when a work zone or special event is present. This factor is multiplied by the expected crash frequency for the segment and intersection, as appropriate. Their product represents the expected crash frequency if the work zone or special event were present for 1 year.

Two adjustment factors are needed for each alternative data set. One factor applies to segment-related crashes, and the other factor applies to intersection-related crashes. The pair of factors is applied to all segments and intersections on the facility, regardless of whether the work zone is on a small portion of the facility or if it extends for the entire length of the facility.

The factor value should include consideration of the effect of the work zone or special event on traffic volume and crash risk. For example, the volume may be reduced due to diversion, and changes to the geometry and signal operation may increase the potential for a crash. To illustrate this concept, consider a work zone that is envisioned to increase crash risk by 100% (i.e., crash risk is doubled) and to decrease traffic volume by 50% (i.e., volume is halved). In this situation, the crash frequency adjustment factor is 1.0 (2.0×0.5). The analyst's experience with similar types of work zones or special events should be used to determine the appropriate adjustment factor value for the subject facility.

Data Entry

This section describes the data entry process for the tool. Data are entered primarily in the Set Up worksheet. However, the default values and calibration factors in the calibration worksheets can also be changed if appropriate local data are available. Guidance for using the USCE to create an HCM data set is provided in the section titled "Urban Streets Computational Engine Data Entry."

The analyst should confirm that he or she has enabled macro operation in the workbook before starting the data entry process. The procedure for enabling macros is described in the section "Getting Started."

Data Entry Basics

The worksheet cells are used for data entry. Some cells accept numeric data, which can be typed in directly using the keyboard. Some cells provide a drop-down list of text choices. In this case, the analyst should use the mouse pointer to select the applicable choice.

If a numeric entry is not within an allowed range, or if it does not match one of the drop-down list of text choices, then a message box indicating "Out of Range!" is displayed. The analyst can click Retry and reenter the data, or click Cancel and return to the cell's previous content.

Set Up Worksheet

The Set Up worksheet is divided into six sections. The first five sections are used for data entry; the sixth is used to display advisory information.

General Information

The organization of this section is shown in Figure B.2. The Location data entry field is used to describe the project being evaluated, and the Analyst data entry field is used to identify the person conducting the evaluation. These entries are not used by the reliability methodology. They are optional data entry fields that will accept any desired combination of numeric and character data.

The Nearest City data are entered using a drop-down list. This information is used to identify the typical weather conditions for the subject facility. If the weather for the city nearest to the subject site does not adequately describe the weather of the subject site, then the Calib-Weather worksheet can be modified to include weather statistics for the location of interest. The analyst will need to select for replacement one of the cities shown in the worksheet (i.e., one row). The data in this row are then deleted. Next, the city name should be entered in each of Columns C, U, AM, BE, and BW. The monthly average weather statistics for this city should be entered in the corresponding row for each of the five tables.

The Functional Class data are entered using a drop-down list. This information is used to estimate the traffic volume during each of the various scenarios that comprise the RRP. Functional class defines the month-of-year and hour-of-day volume adjustment factors. The Calib-Demand worksheet can be modified if the analyst has factors that are more representative of the subject site.

Input File Names

The Path data entry field is used to define the location of the HCM data set files. This path also defines the location of the data sets generated by the tool. The path must exist on the storage drive (i.e., it will not be created if it does not exist).

The Base data entry field is used to record the file name of the base data set. This file includes turn-movement volumes for each intersection on the facility. The date that these volumes were counted is entered in the Date of Traffic Count field. If this field is left blank, then the volumes are assumed to represent an average day's volume, as may be derived from the AADT volume. Similarly, the starting hour of this count is entered in the Starting Hour of Count field. A starting hour of 1:30 p.m. is entered as 13.5.

Seven rows are available to enter alternative data sets; one row is used to describe each data set. The file name is entered in the Alternative field. A description of the work zone or special event is provided in the Description field. The information in this field is not used by the reliability methodology; it is an optional data entry field that will accept any desired combination of numeric and character data.

The Basis of Alternative Traffic Volume field is used to indicate the date associated with the volumes in the alternative data sets. The analyst can specify the same date as that associated with the base data set. Alternatively, if the field is left blank, then the volumes are assumed to represent an average day volume.

Time Period Data

The data entry fields associated with the analysis time period data are shown in Figure B.3.

The Analysis Period data entry field allows the analyst to indicate a 0.25-h or a 1.0-h analysis period. If a 0.25-h analysis period is selected, then additional input data are needed in the Supplemental Input Data section (discussed later in this section). The tool monitors the Analysis Period entry. Depending on the value entered, the tool will either highlight the appropriate data entry cells in the Supplemental Input Data section with a light-blue background, or it will change the cells to white, which indicates that the supplemental data are not needed.

There are two cells for the Study Period data entry. The data in the first cell define the starting hour of the study period in terms of hours since midnight. A value of 0 corresponds to midnight; a value of 13.5 corresponds to 1:30 p.m. The data in the second cell indicate the duration of the study period.

The Reliability Reporting Period field also has two cells. The data in the first cell define the start date using a month-day-year format. The data in the second cell indicate the duration of the RRP.

There are two cells for each alternative data set. The data in the first cell define the start date of the work zone or special event using a month-day-year format. The data in the second cell indicate the duration of the RRP.

The Days of Week Considered data entry consists of seven cells. One cell is associated with one day of the week. A Yes or No is entered in each cell. If Yes is entered, then the associated day is considered in the reliability analysis.

Crash Data

The data entry fields associated with the crash data are shown in Figure B.4.

The upper portion of the Crash Data section is used to enter the crash frequency for the segments and intersections that comprise the urban street facility. The eight light-blue cells in the upper-left portion are used to enter the segment crash frequency. One cell is associated with each segment. The segment numbers are defined when the base data set is created using the USCE.

Time Period Data							
Time Periods		**Start**	**Duration**	**End**	**Time Period Checks**		
Analysis period, h:			0.25		ok		
Study period, h:		7	3	10	ok		
Reliability reporting period, day:		1/1/2011	365	12/31/2011	ok		
Alternative 1 operating period, day:		1/2/2011	3	1/4/2011	ok		
Alternative 2 operating period, day:				.	.		
Alternative 3 operating period, day:				.	.		
Alternative 4 operating period, day:				.	.		
Alternative 5 operating period, day:				.	.		
Alternative 6 operating period, day:				.	.		
Alternative 7 operating period, day:				.	.		
Days of week	Sunday	Monday	Tuesday	Wednesday	Thursday	Friday	Saturday
considered:	No	Yes	Yes	Yes	Yes	Yes	No

Figure B.3. Set Up worksheet: Time Period Data section.

Crash Data ↖see note					
Segment Number	**Segment Boundary Intersections**	**Crash Frequency cr/year**		**Intersection Number**	**Crash Frequency, cr/year**
				1	32
1	1 to 2	15		2	33
2	2 to 3	16		3	34
3	3 to 4	17		4	35
4	4 to 5	18		5	36
5	5 to 6	19		6	37
6	6 to 7	20		7	38
7	7 to 8			8	
8	8 to 9			9	
Work Zone and Special Event Crash Frequency Adjustment Factors ↖see note					
	Segment	**Intersection**			
Alternative 1	1.1	1.2			
Alternative 2					
Alternative 3					
Alternative 4					
Alternative 5					
Alternative 6					
Alternative 7					

Figure B.4. Set Up worksheet: Crash Data section.

The nine light-blue cells in the upper-right portion are used to enter the intersection crash frequency. One cell is associated with each signalized intersection. The intersection numbers are defined when the base data set is created using the USCE.

The lower portion of the Crash Data section is used to enter the work zone and special event crash frequency adjustment factors. Two data entry cells are available for each alternative data set. One cell is used to enter the adjustment factor for all segments on the facility. The second cell is used to enter the adjustment factor for all intersections on the facility.

Supplemental Input Data

The data entry fields associated with the supplemental data are shown in Figure B.5. Light-blue data entry fields are available when a 0.25-h analysis period is used. A Yes or No is entered in the Randomize Demand data entry field. If Yes is entered, then nine light-blue cells are displayed. The peak hour factor is entered for each intersection in the associated light-blue cell. If No is entered, then these cells are white, and no additional data entry is necessary.

Three Seed Numbers data entry cells are shown on the right side of the Supplemental Input Data section. One seed number data entry cell is used for weather predictions, another for demand prediction, and a third for incident prediction. A unique sequence of events is predicted for a given seed number.

One, two, or three of the seed numbers can be changed to generate a different set of conditions, if desired. For example, if the seed number for weather events is changed, then a new series of weather events is created and, to the extent that weather influences incident occurrence, a new series of incidents is created. Similarly, the seed number for demand variation can be used to control whether a new series of demand volumes is created, and the seed number for incidents can be used to control whether a new series of incidents is created.

When evaluating different improvement strategies, it is likely that the analyst will use one set of seed numbers as a variance-reduction technique. In this application, the same seed numbers are used for the evaluation of each strategy. With this approach, the results from an evaluation of one strategy can be compared with those from an evaluation of another strategy. Any observed difference in the results can be attributed to the changes associated with the strategy (i.e., they are not due to random changes in weather or incident events among the evaluations).

Supplemental Input Data						
Randomize demand among 15-min analysis periods within hour:			Yes		**Seed Numbers**	
Intersection Number	**PHF**	**Intersection Number**	**PHF**		Weather:	82
1	0.99	6	0.96		Demand:	11
2	0.92	7	0.97		Incident:	63
3	0.93	8				
4	0.94	9				
5	0.95					

Figure B.5. Set Up worksheet: Supplemental Input Data section.

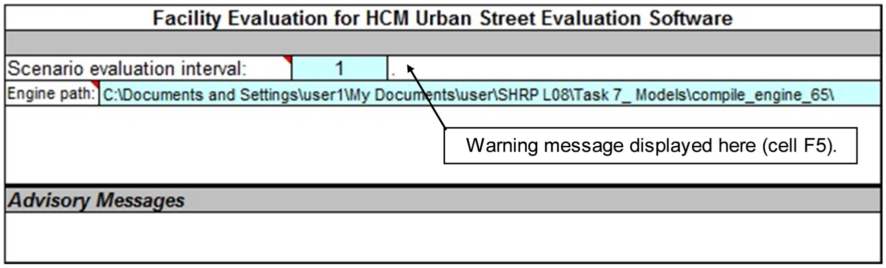


Figure B.6. Facility Evaluation worksheet.

One evaluation of each strategy using the same set of random-number seeds is called a replication. Multiple replications are needed to quantify the best estimate of the desired performance measure and its associated confidence interval. Each replication would use a different set of seed values. The Analysis worksheet can be used to compare strategies based on two or more replications. This worksheet uses statistics to compare strategies by quantifying (1) the expected change in performance and (2) the level of confidence that can be placed in a claim that one strategy has a different performance than another.

Advisory Messages

The Advisory Messages section is used to report software-generated warning messages to the analyst. The analyst should check this section after the calculations are completed and before moving on to the Facility Evaluation worksheet. If a message is shown, then the analyst should make the requested corrections and repeat the calculations. If this section is blank, then the analyst can move to the Facility Evaluation worksheet and continue the analysis.

Facility Evaluation Worksheet

The Facility Evaluation worksheet has two data entry fields: the Scenario Evaluation Interval field and the Engine Path field (see Figure B.6).

The Scenario Evaluation Interval uses units of days; it is used to minimize the total evaluation time. The analyst can chose to evaluate every scenario for every day (i.e., enter 1). Alternatively, the analyst can chose to evaluate every scenario for every other day (i.e., enter 2). This choice will reduce the evaluation time by a factor of two (by evaluating only one-half of the scenarios). More generally, the analyst can specify any integer number for the evaluation interval. The value that is entered is checked by the tool to ensure that it will not bias the results or produce an unacceptably small sample. If the check indicates that an unacceptable outcome will occur, then a warning message is displayed just to the right of the cell specifying the number of days in the scenario evaluation interval (i.e., in cell F5 as shown in Figure B.6).

The Engine Path data entry field is used to define the location of the executable file that implements the HCM2010 urban streets methodology.

Performance Summary Worksheet

Three drop-down lists in the Performance Summary worksheet accept input from the analyst (see Figure B.7). Information input in these lists is used to determine the scope of the performance summary.

The Direction of Travel list is used to indicate which of the two travel directions is of interest to the analysis. One direction is the eastbound (EB) or northbound (NB) travel direction; the other is the westbound (WB) or southbound (SB) direction. The USCE defines the EB or NB directions to coincide with Phase 2 of the National Electrical Manufacturers Association (NEMA). It defines the WB or SB directions to coincide with NEMA Phase 6.

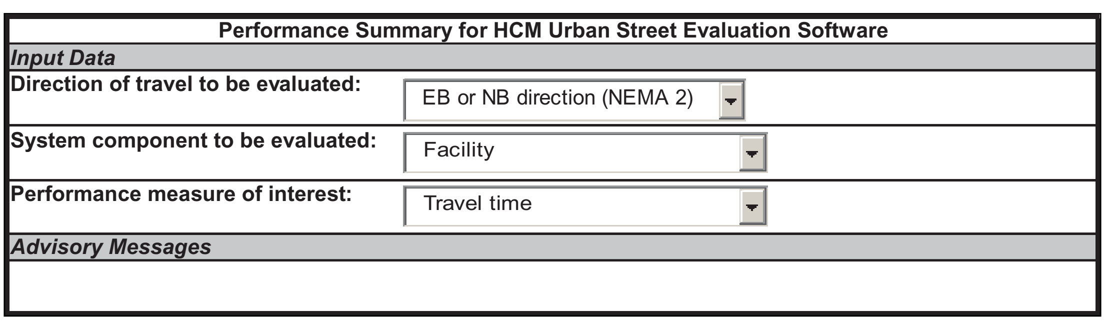


Figure B.7. Performance Summary worksheet.

The System Component list is used to indicate whether the performance summary is based on the entire facility or just one segment. Any one of the segments can be individually selected to facilitate a detailed examination of segment performance.

The Performance Measure list is used to specify the following performance measures of interest:

- Travel time;
- Travel speed;
- Stop rate;
- Running time;
- Through delay; and
- Total delay.

With one exception, all the measures in the list above describe the performance of the major-street through movement. The last measure describes the total delay (in vehicle hours) at one or more intersections. If Facility is selected as the system component, then the total delay is computed for all intersections. If Segment is selected as the system component, then the total delay is computed for the intersection at the end of the segment in the direction of travel evaluated. For a given intersection lane group, total delay is computed as the product of the analysis period duration, lane-group volume, and lane-group control delay. The lane-group total delay is computed for all intersection lane groups, and then these values are added to obtain the intersection total delay.

If travel time is the selected performance measure, then the vehicle miles traveled (VMT) is computed. VMT is computed for each segment and in each scenario and added for all segments on the facility and all scenarios in the RRP. This statistic describes overall facility utilization for the RRP.

If travel time is the selected performance measure, then the reliability rating is also computed. The reliability rating describes the percentage of VMT on the facility associated with a travel time index (TTI) less than 2.5. A facility that satisfies this criterion during a given scenario is likely to provide a level of service D or better for that scenario. The TTI is computed using the average travel speed (as opposed to a percentile value). The TTI and VMT are computed for each segment in each scenario. The VMT for those segments and scenarios with a TTI less than 2.5 is summed for all segments on the facility and all scenarios in the RRP. This VMT is then used to compute the reliability rating.

Results Review and Interpretation

This section describes the output data provided by the tool. These data are provided in the following four worksheets:

- Performance Summary;
- Weather;
- Demand; and
- Incident.

Performance Summary Worksheet

The output data in the Performance Summary worksheet are specific to the direction of travel, system component, and performance measure requested by the analyst. They are displayed in three locations in the worksheet. The first location, in the upper-left portion of the worksheet, describes summary statistics of the performance measure distribution. These statistics are displayed near the top of the Performance Summary worksheet in the Summary Statistics section (see Figure B.8). The data shown correspond to the facility travel time for a 1-year RRP and a 3-h study period.

Three columns of data are shown in the Summary Statistics section. The base free-flow speed and base free-flow travel time are shown in the left column. These statistics are always reported, regardless of the performance measure requested by the analyst. They are relevant to the evaluation of travel time data and the calculation of a TTI.

The middle and right columns of data in the Summary Statistics section summarize the performance measure requested by the analyst. The middle column lists the average, standard deviation, skewness, and median statistics. The right column lists a selected set of percentile values.

The second location for output data is just to the right of the Summary Statistics section. It includes a figure that shows the frequency distribution of the requested performance measure. An example frequency distribution is shown in Figure B.9. This figure shows the facility travel time distribution corresponding to the statistics summarized in Figure B.8.

The third location for output data, just below the Summary Statistics section, lists in chronological order the performance measure for each analysis period. A sample of these data for the performance measure of facility travel time is shown in Figure B.10. Measures are listed by date, month, day of week,

Summary Statistics					
Scenario evaluation interval:	1	Average:	443.74	5th percentile:	344.97
Base free-flow speed, mi/h:	41.08	Standard deviation:	309.41	10th percentile:	347.63
Base free-flow travel time, s:	262.90	Skewness:	7.20	80th percentile:	412.72
Reliability rating:	93.2	Median:	371.85	85th percentile:	431.13
Total vehicle-miles travel (1,000's):	2260	Number of obs.:	3120	95th percentile:	783.82

Figure B.8. Performance Summary worksheet: Summary Statistics section.

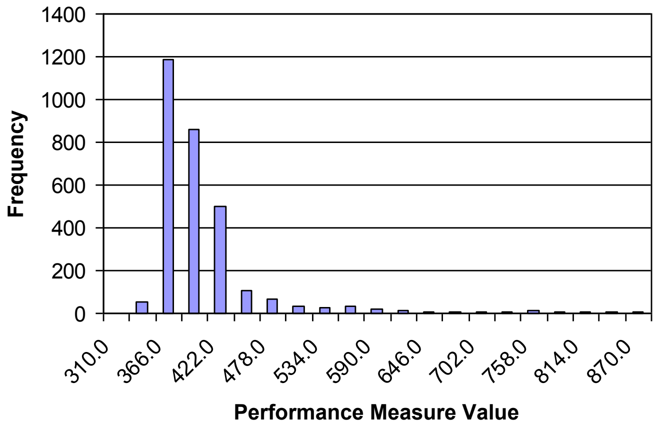


Figure B.9. Performance Summary worksheet: frequency distribution.

and start time of the analysis period. The day of week is numeric; Sunday is Day 1.

Weather Worksheet

The Weather worksheet provides supplemental output data (see Figure B.11).

Data are located in two sections of the Weather worksheet. The top portion of the worksheet lists the weather statistics for the city nearest to the subject facility. These statistics were read from the Calib-Weather worksheet and reported back to the Weather worksheet for the analyst's convenience. One set of weather statistics is provided for each month of the year. The statistics for October, November, and December are not shown in Figure B.11 so that the figure text could be kept at a readable size.

The second location for output data, just below the weather statistics, lists in chronological order the predicted weather conditions for each analysis period. A sample of these data is shown in the lower portion of Figure B.11. The first six columns define the date and time of the analysis period. The remaining columns describe the weather conditions. A snow event is shown to occur starting at 7:00 a.m. on January 5, 2011. The precipitation amount in the second to the last column describes the liquid equivalent of the snowfall rate.

Demand Worksheet

The Demand worksheet provides supplemental output data (see Figure B.12) that consist of the demand volume adjustment factors that were read from the Calib-Demand worksheet. These factors represent the distribution of volume by hour of day, day of week, and month of year.

Data are located in two sections of the Demand worksheet. The upper portion of the worksheet lists the adjustment factors that are applicable to the base data set and the alternative data sets. These factors are used to convert the turn-movement volumes in the data set to AADT volume estimates. The information in Figure B.12 indicates that the turn-movement volumes in the base data set are based on 1-h counts taken on January 4, 2011, starting at 7:00 a.m. The data entry fields for this date and time are provided in the Set Up worksheet, as shown in Figure B.2.

Period Number	Day	Date	Month	Day of Week	Start Time (hr)	Perf. Measure (s)
1	3	1/3/2011	1	2	7.00	366.6
2	3	1/3/2011	1	2	7.25	367.5
3	3	1/3/2011	1	2	7.50	363.3
4	3	1/3/2011	1	2	7.75	367.1
5	3	1/3/2011	1	2	8.00	351.5
6	3	1/3/2011	1	2	8.25	356.2
7	3	1/3/2011	1	2	8.50	348.2
8	3	1/3/2011	1	2	8.75	347.3
9	3	1/3/2011	1	2	9.00	332.8
10	3	1/3/2011	1	2	9.25	333.0
11	3	1/3/2011	1	2	9.50	333.9
12	3	1/3/2011	1	2	9.75	332.9
13	4	1/4/2011	1	3	7.00	369.8
14	4	1/4/2011	1	3	7.25	365.7
15	4	1/4/2011	1	3	7.50	370.5
16	4	1/4/2011	1	3	7.75	368.5
17	4	1/4/2011	1	3	8.00	350.5
18	4	1/4/2011	1	3	8.25	348.6
19	4	1/4/2011	1	3	8.50	349.3

Figure B.10. Performance Summary worksheet: analysis period results.

Location: LINCOLN, NE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
Normal precipitation, in/month	0.67	0.66	2.21	2.9	4.23	3.51	3.54	3.35	2.92
Snowfall, in/month	6.6	6	5.7	1.5	0.1	0	0	0	0
Days with precip./month	5	5	8	9	11	9	9	8	7
Average temp, degrees	22.4	28.3	39.4	51.2	62	72.7	77.8	75.4	66
Precip. per event, in/event	0.13	0.13	0.28	0.32	0.38	0.39	0.39	0.42	0.42

Total analysis periods with weather effects:	144
Total analysis periods:	3120
Total periods with rain events:	76
Total periods with snow events:	33

Period Number	Day	Date	Month	Day of Week	Start Time (h)	Rain Amount (in/h)	Wet Pavement	Snow Amount (in/h)	Snow or Ice on Pavement	Precip. Amount (in/h)	Total
19	4	1/4/2011	1	3	8.50	0.00	no	0.00	no	0.00	
20	4	1/4/2011	1	3	8.75	0.00	no	0.00	no	0.00	
21	4	1/4/2011	1	3	9.00	0.00	no	0.00	no	0.00	
22	4	1/4/2011	1	3	9.25	0.00	no	0.00	no	0.00	
23	4	1/4/2011	1	3	9.50	0.00	no	0.00	no	0.00	
24	4	1/4/2011	1	3	9.75	0.00	no	0.00	no	0.00	
25	5	1/5/2011	1	4	7.00	0.00	no	0.79	YES	0.08	1
26	5	1/5/2011	1	4	7.25	0.00	no	0.79	YES	0.08	1
27	5	1/5/2011	1	4	7.50	0.00	no	0.79	YES	0.08	1

Figure B.11. Weather worksheet.

The lower portion of the Demand worksheet lists in chronological order the adjustment factors used for each analysis period. These factors are used to convert the AADT estimates into turn-movement volumes corresponding to the specific analysis period.

Incident Worksheet

The Incident worksheet provides supplemental output data (see Figure B.13).

The Incident worksheet lists in chronological order the incident type and location for each analysis period. One group of 12 columns is dedicated to each intersection and to each segment on the facility. For a given intersection or segment, the 12 columns collectively describe the incident type, lane location, and severity. Each row of the worksheet corresponds to one analysis period. If all the cells are blank for a given analysis period (i.e., row), then no incident was predicted to occur. If a cell has a value, then an incident is indicated to exist for that time period.

Location: LINCOLN, NE
Functional Class: Urban Principal Arterial

	Date	Month	Day of Week	Start Time (h)	Hr of Day Factor	Day of Wk Factor	Month of Yr Factor
Date of traffic count	1/4/2011	1	3	7	0.071	0.980	0.831
Basis of alt. traffic vol.:	Ave.day			7	0.059	1.000	1.000

Day of Week: 1:Sun | 2:Mon | 3:Tue | 4:Wed | 5:Thr | 6:Fri | 7:Sat

Total analysis periods with unique factors:	720
Total analysis periods:	3120

Period Number	Day	Date	Month	Day of Week	Start Time (h)	Hr of Day Factor	Day of Wk Factor	Month of Yr Factor	Total
1	3	1/3/2011	1	2	7.00	0.071	0.980	0.831	1
2	3	1/3/2011	1	2	7.25	0.071	0.980	0.831	1
3	3	1/3/2011	1	2	7.50	0.071	0.980	0.831	1
4	3	1/3/2011	1	2	7.75	0.071	0.980	0.831	1
5	3	1/3/2011	1	2	8.00	0.058	0.980	0.831	1
6	3	1/3/2011	1	2	8.25	0.058	0.980	0.831	1
7	3	1/3/2011	1	2	8.50	0.058	0.980	0.831	1
8	3	1/3/2011	1	2	8.75	0.058	0.980	0.831	1
9	3	1/3/2011	1	2	9.00	0.047	0.980	0.831	1

Figure B.12. Demand worksheet.

Total analysis periods with incidents:		156		
Total analysis periods:		3120		Expected
	Segment	Intersection	Total	Total
Total crash incidents:	4	13	17	11.6
Total non-crash incidents:	6	27	33	24.6

						Intersection 1											
						Crash						Non-Crash					
						One lane		Two+ Lanes		Shoulder		One lane		Two+ Lanes		Shoulder	
Period				Day of	Start	Fatal or Inj	PDO	Fatal or Inj	PDO	Fatal or Inj	PDO	Brkdwn	Other	Brkdwn	Other	Brkdwn	Other
Number	Day	Date	Month	Week	Time	2	0	8	0	4	0	5	0	4	0	4	0
1093	130	5/10/2011	5	3	7.00												
1094	130	5/10/2011	5	3	7.25												
1095	130	5/10/2011	5	3	7.50												
1096	130	5/10/2011	5	3	7.75												
1097	130	5/10/2011	5	3	8.00			2									
1098	130	5/10/2011	5	3	8.25			2									
1099	130	5/10/2011	5	3	8.50			2									
1100	130	5/10/2011	5	3	8.75			2									
1101	130	5/10/2011	5	3	9.00			2									
1102	130	5/10/2011	5	3	9.25			2									
1103	130	5/10/2011	5	3	9.50			2									
1104	130	5/10/2011	5	3	9.75			2									
1105	131	5/11/2011	5	4	7.00												

Figure B.13. Incident worksheet.

The incident location is indicated by determining in which group of 12 columns the nonblank cell exists. That is, if a group of 12 columns has one or more cells with a value in it, then this condition indicates that the associated intersection or segment was predicted to experience an incident. If the location is an intersection, then the cell value indicates the intersection approach location of the incident. The cell value represents the NEMA number of the phase that serves the affected intersection approach. If the location is a segment, then the cell value indicates the direction of travel associated with the incident. The value is the NEMA number of the phase that serves the direction of travel.

The NEMA phase-numbering scheme for the subject facility is defined in the USCE and communicated to the tool using the HCM data set. The USCE defines the EB or NB directions to coincide with NEMA Phase 2. It defines the WB or SB directions to coincide with NEMA Phase 6. It defines the NB or WB directions to coincide with NEMA Phase 8. It defines the SB or EB directions to coincide with NEMA Phase 4.

For example, the data in Figure B.13 indicate that a crash occurred on the Phase 2 approach of Intersection 1 at 8:00 a.m. on May 10, 2011. The crash was a fatal or injury crash that blocked two or more lanes for a 2-h period.

Supplemental Guidance for Assigning Reported Crashes

The reliability methodology requires an estimate of the expected average crash frequency for intersections and for segments on the facility. One source of this estimate is reported crash history data. However, these crashes must be properly assigned to the location of their occurrence. The assignment process requires differentiation of each crash as either an intersection-related crash or a segment-related crash. This section describes the crash assignment procedure recommended in the *Highway Safety Manual* (American Association of State Highway and Transportation Officials 2010).

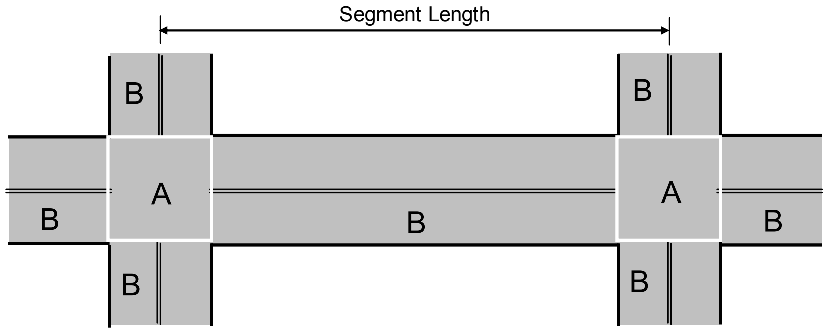
Intersection crashes include crashes that occur at an intersection (i.e., within the curb limits) and crashes that occur on the intersection legs and are intersection related. All crashes that are not classified as intersection or intersection-related crashes are considered to be segment-related crashes.

Figure B.14 illustrates the method used to assign crashes to segments or intersections. As shown, all crashes that occur within the curb line limits of an intersection (i.e., Region A) are assigned to that intersection.

Crashes that occur outside the curb line limits of an intersection (i.e., Region B) are assigned to either the segment on which they occur or an intersection, depending on their characteristics. Region B represents the roadway between two intersections. Crashes that are classified on the crash report as intersection related or have characteristics consistent with an intersection-related crash are assigned to the intersection to which they are related; such crashes would include rear-end crashes related to queues on an intersection approach. Crashes that occur between intersections and are not related to an intersection are assigned to the roadway segment on which they occur.

In some jurisdictions, crash reports include a field that allows the reporting officer to designate the crash as intersection related. When this field is available on the crash reports, crashes should be assigned to the intersection or the segment based on the way the officer marked the field on the report.

In jurisdictions where there is not a field on the crash report that allows the officer to designate crashes as intersection related, the characteristics of the crash may be considered to help determine whether the crash should be assigned to the intersection or the segment. Other fields on the report, such as crash type, number of vehicles involved, contributing circumstances, weather condition, pavement condition,



A All crashes that occur within this region are classified as intersection crashes.
B Crashes in this region may be segment or intersection related, depending on the characteristics of the crash.

Figure B.14. Definition of roadway segments and intersections.

traffic control malfunction, and sequence of events can provide helpful information in making this determination. If the officer's narrative and a crash diagram are available, they can also assist in making the determination of a crash's intersection relationship.

The following crash characteristics are indicative of an intersection-related crash:

- A rear-end crash in which both vehicles were going straight approaching an intersection or in which one vehicle was going straight and struck a stopped vehicle; and
- A crash in which the report indicates a signal malfunction or improper traffic control at the intersection contributed to the crash.

Urban Streets Computational Engine Data Entry

This section describes the data entry process for the USCE, which implements the urban streets methodology described in Chapter 16 of the HCM2010. The 10 input worksheets in the USCE are as follows:

- Set Up: general input data to describe overall characteristics of the facility and start calculations;
- Intersection 1: input data describing the first signalized intersection encountered in the EB or NB travel direction;
- Segment 1: input data describing the segment located between Intersections 1 and 2;
- Segment 2: input data describing the segment located between Intersections 2 and 3;
- Segment 3: input data describing the segment located between Intersections 3 and 4;
- Segment 4: input data describing the segment located between Intersections 4 and 5;
- Segment 5: input data describing the segment located between Intersections 5 and 6;
- Segment 6: input data describing the segment located between Intersections 6 and 7;
- Segment 7: input data describing the segment located between Intersections 7 and 8; and
- Segment 8: input data describing the segment located between Intersections 8 and 9.

The segment and intersection numbers are defined sequentially in the EB or NB direction of travel. The data entered in each segment worksheet describe both the road between the intersections and the downstream signalized intersection. The data entered in each segment worksheet are identical, so only the data entry for the Segment 1 worksheet will be described in this section. If the facility has fewer than eight segments, then data are entered only for those segments that exist.

Many input values and parameters are associated with the USCE. The meaning of each value or parameter is described in Chapter 17 or 18 of the HCM2010. The data entry cell for each parameter is populated with a default value in the USCE.

After the data are entered, the analyst should return to the Set Up worksheet to save the data in an HCM data set. Clicking the button labeled Write Data to File saves the data to a file. This button is located in the upper-right corner of the worksheet. The next section describes where the analyst can enter the file name.

Set Up Worksheet

The data entry fields in the Set Up worksheet are divided into six sections. The input data fields in each section are described in the following paragraphs.

General Information

The General Information section shown in Figure B.15 is located near the top of the Set Up worksheet.

General Information			
Location: Texas Avenue, Austin, Texas		**Analysis Period:**	7:15 am to 7:30 am
File name: C:\Documents and Settings\TexasAM2		**Analyst:**	JHD
Urban Street Parameters			
Start-up lost time (l_1), s	2.0	Stored vehicle lane length, ft	25
Extension of effective green, s	2.0	Number of calculation iterations	15
Analysis time period (T), h	0.25	Length of left-turn bay (access point), ft	250
Critical merge headway, s	3.7	Right-turn equivalency factor (signalized)	1.18
Deceleration rate (access point), ft/s^2	6.7	Sneakers per cycle, veh:	2.0
Right-turn speed (access point), ft/s	20	Base saturation flow rate, pc/h/ln	1900
Deceleration rate (signal), ft/s^2	4.0	Distance between stored vehicles, ft	8.0
Acceleration rate, ft/s^2	3.5	Left-turn equivalency factor (signalized)	1.05
Headway of bunched vehicle stream, s/veh	1.5	Critical headway for major left (access pt.), s	4.1
Maximum headway in a platoon, s/veh	3.6	Follow-up headway for major left (access pt.), s	2.2
Stop threshold speed, mph	5.0	Right-turn equivalency factor (access point)	2.2

Figure B.15. USCE Set Up worksheet: General Information and Urban Street Parameters sections.

The Location, Analysis Period, and Analyst data entry fields are used to describe the project being evaluated. These entries are not used by the reliability methodology. They are optional data entry fields that will accept any desired combination of numeric and character data.

The File Name data entry field is used to define the path location and name of the HCM data set files. The path must exist on the storage drive (i.e., it will not be created if it does not exist).

Urban Street Parameters

The data entry fields in the Urban Street Parameters section are shown in the lower portion of Figure B.15. Two additional parameters can be entered in the Supplemental Urban Street Parameters section that is located at the bottom of the Set Up worksheet (not shown). Default values are provided for these parameters.

Basic Segment Information

The Basic Segment Information section is shown in Figure B.16. The names of the two streets that bound the segment are listed in Columns 2 and 3. The name of the street crossed when departing the segment in the WB (or SB) direction is shown in Column 2. The name of the street crossed when departing the segment in the EB (or NB) direction is shown in Column 3.

Basic Segment Information							
Segment Number	Cross Street Names		Speed Limit, mph		Through Lanes		Segment Length, ft
	Street to West	Street to East	EB	WB	EB	WB	
1	First Avenue	Second Avenue	35	35	2	2	1800
2	Second Avenue	Third Avenue	35	35	2	2	1800
3	Third Avenue	Fourth Avenue		35			
4	Fourth Avenue	Fifth Avenue					
5	Fifth Avenue	Sixth Avenue					
6	Sixth Avenue	Seventh Avenue					
7	Seventh Avenue	Eighth Avenue					
8	Eighth Avenue	Ninth Avenue					
			No. Intersections:		3	Total, mi:	0.68
Coordination Information							
Travel direction for Movement 2 at all intersections			EB	Cycle Length, s:		100	

Origin–Destination Seed Proportions	Upstream Origin			
Downstream Destination	Cross St. Left Turn	Major St. Through	Cross St. Right Turn	Mid-Seg. Entry
Left Turn	0.02	0.10	0.05	0.02
Through	0.91	0.78	0.92	0.97
Right Turn	0.05	0.10	0.02	0.01
Mid-Segment Exit	0.02	0.02	0.01	0.00

Figure B.16. USCE Set Up worksheet: Basic Segment and Coordination Information sections.

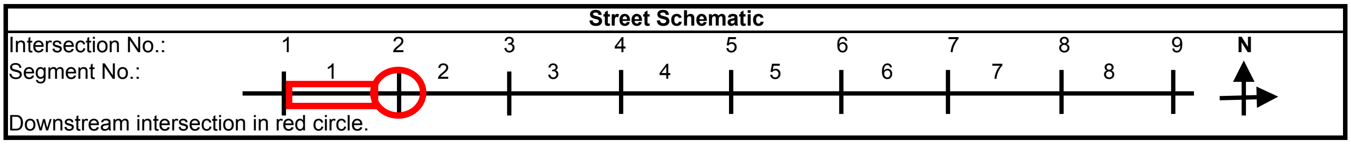


Figure B.17. USCE Set Up worksheet: Intersection and Segment Numbers.

This section is also used to inform the USCE about the number of intersections on the facility. This number is reported in the last line of this section. It is based on the count of segments for which the analyst has provided speed limit, lane, and segment length data in the rows above. The number of intersections is equal to one more than the number of segments.

The data shown in Figure B.16 indicate that there are two segments and three intersections on the subject facility. The speed limit for the WB approach to Intersection 3 (at Fourth Avenue) is 35 mph. This approach is external to the facility (i.e., it serves through vehicles, but it is not on one of the segments). If the facility has eight segments, then the speed limit for the WB approach to Intersection 9 (at Ninth Avenue) is entered in the Segment 8 worksheet. Similarly, the EB approach at Intersection 1 is external to the facility. The speed limit for this approach is entered in the Intersection 1 worksheet. Based on these data entry rules, the reader will note that there will always be one more speed limit entry in the WB column than in the EB column whenever there are fewer than eight segments.

Coordination Information

The Coordination Information section is used to define the direction of travel associated with NEMA Phase 2, which is the same as Movement 2. The direction that is chosen is then used to establish the association between the intersection number, segment number, phase number, and travel direction. A street schematic of the facility is shown in Figure B.17; the intersection and segment numbers are established by the data entry. This schematic is repeated at the top of the Intersection worksheet and at the top of each Segment worksheet.

Origin–Destination Seed Proportions

The data entry fields in the Origin–Destination Seed Proportions section are used to enter the proportion of upstream traffic (by movement) that arrives at each downstream destination movement. The values in these cells represent default values and will not likely need to be modified for most reliability evaluations.

Intersection 1 Worksheet

The data entered in the Intersection 1 worksheet is categorized in three sections. The data entry fields in each section are described in the following paragraphs. There is also an Advisory Messages section near the bottom of this worksheet (not shown) that should be consulted after all data are entered in this worksheet. Advisory messages will be posted when the data entered are contradictory, missing, or out of range.

Signalized Intersection Input Data

The data entered in the Signalized Intersection Input Data section are shown in Figure B.18. This section describes the data entry fields for the individual movements at the intersection. One column is provided for data entry for the left-turn, through, and right-turn movements on each intersection

Signalized Intersection 1												
Signalized Intersection Input Data *(In each column, enter the volume and lanes data. For all other blue cells, enter values only if there is one or more lanes.)*												
Approach	Eastbound			Westbound			Northbound			Southbound		
Movement	L	T	R	L	T	R	L	T	R	L	T	R
Movement number	5	2	12	1	6	16	3	8	18	7	4	14
Volume, veh/h	200	1000	10	200	1000	10	100	500	50	100	500	50
Lanes	1	2	1	1	2	1	1	2	0	1	2	0
Turn bay length, ft	200		200	200		200	200			200		
Sat. flow rate, veh/h/ln	1800	1800	1800	1800	1800	1800	1800	1800		1800	1800	
Platoon ratio	1.000	1.333	1.000	1.000	1.333	1.000	1.000	1.000		1.000	1.000	
Initial queue, veh	0	0	0	0	0	0	0	0		0	0	
Speed limit, mph	35	35	35	35	35	35	35	35	35	35	35	35
Stop line det. length, ft	40			40			40	40		40	40	
Max. allow. hdwy, s/veh	3.9			3.9			3.9	2.9		3.9	2.9	
Opp. rt-turn lane influence	Yes			Yes			Yes			Yes		

Figure B.18. USCE Intersection 1 worksheet: Signalized Intersection Input Data section.

approach. Separate groups of three columns are provided for each of the four intersection approaches.

In each column, the volume and lanes data are entered for the associated movement. For all other blue cells, values are entered only if there is one or more exclusive lanes serving the movement. If two or more movements share a lane, then their combined data are entered in the column for the through movement.

Saturation Flow Rate

The Saturation Flow Rate data entry field is used to describe the adjusted saturation flow rate for every movement with one or more lanes. The value entered should reflect the effect of lane width, heavy-vehicle presence, grade, parking activity, local buses that stop, area type, lane utilization, pedestrian conflicts, and bicycle conflicts. If the movement does not exist, then the cell should be left blank (i.e., cell contents should be deleted).

The procedure in Chapter 18 of the HCM2010 (i.e., Step 4) can be used to estimate the adjusted saturation flow rate. However, for left-turn (or right-turn) movements, the saturation flow rate entered should be based on a left-turn (or right-turn) adjustment factor of 1.0 (i.e., do not adjust for permitted or protected–permitted operation).

If a work zone is present on an intersection approach, then an additional saturation flow rate adjustment factor f_{wz} is computed using Equation B.1. This factor is then multiplied by the saturation flow rate obtained from Chapter 18 of the HCM2010 to obtain the value to be entered in the USCE worksheet. Equations B.2 and B.3 are used to calculate the saturation flow rate adjustment factors for approach width and reducing lanes during work zone presence, respectively.

$$f_{wz} = 0.858 \times f_{wid} \times f_{reduce} \qquad \leq 1.0 \tag{B.1}$$

with

$$f_{wid} = \frac{1.0}{1.0 - 0.0057\,(a_w - 12)} \tag{B.2}$$

$$f_{reduce} = \frac{1.0}{1.0 + 0.0402\,(n_o - n_{wz})} \tag{B.3}$$

where

f_{wz} = saturation flow rate adjustment factor for work zone presence;

f_{wid} = saturation flow rate adjustment factor for approach width;

f_{reduce} = saturation flow rate adjustment factor for reducing lanes during work zone presence;

a_w = approach lane width during work zone (total width of all open left-turn, through, and right-turn lanes plus any setback distance to adjacent curb or work zone traffic control devices), ft;

n_o = number of left-turn and through lanes open during normal operation; and

n_{wz} = number of left-turn and through lanes open during work zone presence.

Equation B.1 produces values less than 1.0 for a wide range of conditions. However, when the approach has many lanes open while a work zone is present (or a few wide lanes), then Equation B.1 can mathematically produce a value that exceeds 1.0. In these few instances, a value of 1.0 is recommended as an upper bound on the factor value.

One factor value is computed for each approach with a work zone present. The computed factor is then used to estimate the saturation flow rate for the through movement, as well as the left- and right-turn movements from exclusive lanes, on this approach.

Platoon Ratio

The Platoon Ratio data entry field is provided for all intersection movements. However, with a few exceptions, the values provided for Movements 2 and 6 will be ignored. One of the exceptions is the EB (or NB) Movement 2 at Intersection 1. This movement is an external movement. Its platoon ratio and speed limit need to be entered in this section.

The other exception is for the WB (or SB) Movement 6 at the last intersection on the facility. This movement is also an external movement. In this case, the platoon ratio for this movement will need to be entered in the Segment worksheet associated with the last intersection.

Phase Sequence and Left-Turn Mode

The Phase Sequence and Left-Turn Mode section is shown in Figure B.19. Drop-down lists are used to describe the phase sequence and left-turn mode for the major street and the cross-street approaches. The left-turn mode can be permitted, protected–permitted, or protected only. The USCE requires

Phase Sequence and Left-Turn Mode			
Major street sequence (movement numbers shown)	5 & 1 left leading	Cross street sequence (movement numbers shown)	3 & 7 left leading
Major street left-turn mode (movement numbers shown)	5/1 Protected-Only	Cross street left-turn mode (movement numbers shown)	3/7 Protected+Permitted

Figure B.19. USCE Intersection 1 worksheet: Phase Sequence and Left-Turn Mode section.

Phase Settings								
Approach	**Eastbound**		**Westbound**		**Northbound**		**Southbound**	
Phase number	5	2	1	6	3	8	7	4
Movement	L	T+R	L	T+R	L	T+R	L	T+R
Lead/lag left-turn phase	Lead	--	Lead	--	Lead	--	Lead	--
Left-turn mode	Prot.	--	Prot.	--	Pr/Pm	--	Pr/Pm	--
Passage time, s	2.0	--	2.0	--	2.0	2.0	2.0	2.0
Minimum green, s	5	--	5	--	5	5	5	5
Yellow + red clear, s	3.0	4.0	3.0	4.0	3.0	4.0	3.0	4.0
Phase split, s	20	35	20	35	20	25	20	25
Recall	No	--	No	--	No	No	No	No
Dual entry	No	Yes	No	Yes	No	Yes	No	Yes
Ref. Phase: 2	Offset, s: 0	Cycle, s: 100	Offset Ref.: End of Green		Force Mode: Fixed			
Enable Simultaneous Gap-Out? Phase Group 1,2,5,6: ☑ Phase Group 3,4,7,8: ☑				Enable Dallas Left-Turn Phasing? Phases 1,2,5,6: ☐ Phases 3,4,7,8: ☐				

Figure B.20. USCE Intersection 1 worksheet: Phase Settings section.

that both approaches on a given street have the same left-turn mode. Phase sequence choices are identified in the following list:

- No exclusive phase;
- Lead/lead left-turn phases;
- Lead/lag left-turn phases; and
- Lag/lag left-turn phases.

Phase Settings

The Phase Settings section is shown in Figure B.20. The phase numbers referenced in this figure correspond to the movement numbers shown in Figure B.18.

Segment 1 Worksheet

The data entered in the Segment 1 worksheet are categorized in five sections. The data entry fields in three of these sections are the same as in the Intersection 1 worksheet, so they will not be discussed in this section. The remaining two sections are described in the following paragraphs.

There is also an Advisory Message section near the bottom of this worksheet (not shown) that should be consulted after all data are entered in this worksheet. Messages will be posted when the data entered are contradictory, missing, or out of range.

Free-Flow Speed Input Data

The Free-Flow Speed Input Data section is shown in Figure B.21. The data entered in this section are used to compute the free-flow speed for the segment. A speed is computed for each direction of travel. The values in the white cells were entered in the Set Up worksheet and are repeated here for convenience.

A Supplemental Segment Data section (not shown) located just to the right of the Free-Flow Speed Input Data section allows the analyst to enter any midsegment delay to through

Free-Flow Speed Computation		
Input Data		
	EB	**WB**
Basic Segment Data		
Number of through lanes that extend the length of the segment:	2	2
Speed limit, mph	35	35
Segment Length Data		
Length of segment (measured stopline to stopline), ft	1800	1800
Width of upstream signalized intersection, ft	50	50
Adjusted segment length, ft	1750	1750
Length of segment with a restrictive median (e.g., raised-curb), ft	0	0
Length of segment with a non-restrictive median (e.g., two-way left-turn lane), ft	0	0
Length of segment with no median, ft	1750	1750
Percentage of segment length with restrictive median, %	0	0
Access Data		
Percentage of street with curb on right-hand side (in direction of travel), %	70	70
Number of access points on right-hand side of street (in direction of travel)	4	4
Access point density, access points/mi	24	24

Figure B.21. USCE Segment 1 worksheet: Free-Flow Speed Input Data section.

Access Point Input Data													
Access	Approach	Eastbound			Westbound			Northbound			Southbound		
Point	Movement	L	T	R	L	T	R	L	T	R	L	T	R
Location,ft	Movement number	1	2	3	4	5	6	7	8	9	10	11	12
600	Volume, veh/h	80	1050	100	80	1050	100	80	0	100	80	0	100
West end	Lanes	0	2	0	0	2	0	1	0	1	1	0	1
1200	Volume, veh/h	80	1050	100	80	1050	100	80	0	100	80	0	100
	Lanes	0	2	0	0	2	0	1	0	1	1	0	1
	Volume, veh/h	0	0	0	0	0	0	0	0	0	0	0	0
	Lanes	0	2	0	0	2	0	0	0	0	0	0	0
	Volume, veh/h	0	0	0	0	0	0	0	0	0	0	0	0
	Lanes	0	2	0	0	2	0	0	0	0	0	0	0
	Volume, veh/h	0	0	0	0	0	0	0	0	0	0	0	0
	Lanes	0	2	0	0	2	0	0	0	0	0	0	0
	Volume, veh/h	0	0	0	0	0	0	0	0	0	0	0	0
East end	Lanes	0	2	0	0	2	0	0	0	0	0	0	0

Figure B.22. USCE Segment 1 worksheet: Access Point Input Data section.

vehicles traveling along the segment that is due to sources other than turns at the access points (as described in the next section). These other sources of delay may include curb parking, pedestrian crossings, double parking, and so forth.

Access Point Input Data

The Access Point Input Data section is shown in Figure B.22. The data entered in this section are used to compute the delay to through movements as a result of vehicles turning from the major street into an access point. The access points described in this section are sufficiently busy that they are likely to result in some delay to the major-street through movement.

Data for a maximum of six access points can be entered. These data are entered in order from top to bottom as they occur in an EB or NB direction of travel.

Access point location represents the distance measured from the stop line of the upstream signalized intersection to the equivalent stop line at the downstream access point in the subject direction of travel.

If several low-volume access points exist and none are going to be entered as a separate access point, they can be combined into one surrogate access point. The volume for each minor movement at the surrogate access point should equal the sum of the corresponding minor movements for all access points being combined. The location of the surrogate access point should represent the average of the distances to each of the individual access points that were combined.

Software Setup and File Description

This section describes the software and data files associated with the USRE workbook and summarizes the process for setting up the file structure for a typical application.

Software Description

The software used for a reliability evaluation consists of two Excel workbooks and one executable file. These files are described in this section.

USRE Workbook

The USRE workbook supports the evaluation of a facility's reliability of service in terms of its operational performance over an extended time period. The software is distributed as a MS Excel workbook using VBA programming language. This workbook is used to guide the reliability evaluation and implement the reliability methodology. It interacts with other software and creates data files as needed for the evaluation. Guidelines for using this workbook are provided below.

The file name for this workbook is USRE-XY.xls.

The letters X and Y are used to convey the software version.

USCE Workbook

The USCE workbook implements the urban streets methodology described in Chapter 16 of the HCM2010. This workbook is used to create the HCM data set that includes the input data needed to evaluate an urban street facility using the HCM.

The file name for this workbook is C17_A06-4_G06-4_V2010_L08.xls.

L08 is added to the file name to indicate that this workbook is an enhanced version of the HCM workbook implementing the methodology in HCM2010, Chapter 17. The workbook has been enhanced to explicitly model the effect of spillback on intersection operation.

HCM Executable

This executable file is a compiled version of the VBA code in the USCE workbook. It is called the USRE workbook, and it is used to evaluate the data set associated with each scenario. It is compiled to minimize the time required to evaluate a data set.

The file name for this file is engine17.exe.

The path to this engine is input to the USRE workbook in the Facility Evaluation worksheet.

Data File Description

The three data files associated with the use of the USRE workbook are described in this section.

HCM Data Set File

The HCM data set files are created using the USCE workbook. The base data set is used to describe the urban street when there are no work zones or special events present. One or more alternative data sets are used to describe the urban street when a work zone, special event, or both are present.

The file name for this file is specified by the analyst using any characters acceptable to the Windows operating system. The file extension is ".txt".

The base and alternative data sets are created by the analyst and saved in a folder. The path to this folder is input by the analyst in the Set Up worksheet of the USRE workbook.

Scenario Data Set File

The USRE workbook creates one scenario data set file for each scenario. This data set is created from the HCM data set, but it is modified to reflect the demand levels, speed, and saturation flow rate, as they may be influenced by the events predicted for the specific scenario.

The file name for this file is yyyymmdd-HHhh.txt.

The file name describes the date and time associated with the scenario. The letters yyyy are used to indicate the year, mm indicates the month, and dd indicates the day of the month. HH indicates the hour associated with the start of the scenario (in military time); hh indicates the percentage of an hour associated with the start of the scenario. Values used are 00%, 25%, 50%, and 75%, which correspond to 00, 15, 30, and 45 min, respectively. For example, 1825 indicates the scenario start time is 6:15 p.m.

The scenario data set files are saved in the same folder as the HCM data set file.

Scenario Output File

The USRE workbook creates one scenario output file for each scenario. This data set contains the output from the HCM executable file; that is, it contains the predicted delay and queue length at each intersection, as well as the travel time and travel speed for each segment and for the overall facility.

The file name for this file is yyyymmdd-HHhh.out.

The file naming convention is the same as that for the scenario data set file as described in the previous section.

The scenario data set files are saved in the same folder as the HCM data set file.

The file is a comma-delimited text file and can be read using any word processing software program (e.g., Notepad). It can also be opened in an Excel worksheet. Sample content of this file is shown in Figure B.23. The units are shown on the right-hand side of the figure for each variable listed. The text for the units is shown here for convenience; it is not included in the text file.

The first three entries in the scenario output file shown in Figure B.23 are defined as follows:

- FileName: the first line lists the name of the subject scenario output file.
- NbrSegments: the number of segments for which data are provided in the file. The data for Segment 1 begin in the third row (the data for Segment 2 are not shown).
- SEGMENT: the values in this section describe the performance of the through movement on Segment 1. Each value represents an average for the analysis period. This section is repeated for each numbered segment.

The fourth line identifies the two vehicle movements (by number) representing the major-street through movements. Each movement is associated with one travel direction along the major street. Movement 2 (i.e., EB or NB) is listed first. Movement 6 (i.e., WB or SB) is listed second. This is a header line because it defines the order of presentation for the values listed in the subsequent rows for the segment. Specifically, the first value listed corresponds to Movement 2, and the second value listed corresponds to Movement 6. For example, the row with the segment through delay (SegThruDelay) shows 20.369 seconds per vehicle (s/veh) for Movement 2 travel direction and 20.043 s/veh for Movement 6 travel direction. The movement numbers are shown in Figure B.24.

Other entries in the scenario output file shown in Figure B.23 are defined as follows:

- SegLength: This is the length of the segment, which is the same in each direction of travel.
- SYSTEM: The values in this section describe the performance of the through movement on the facility. They are computed from the through movement values for each segment. Each value represents an average for the analysis period.
- NbrIntersections: The number of intersections for which data are provided in the file. The data for Intersection 1 begin with the next row (the data for Intersections 2 and 3 are not shown).

```
"FileName:","TexasAM2.out"
"NbrSegments",2
"SEGMENT:",1
" 02 06"
"SegLength",1800.............................................................................. (ft)
"SegBaseFreeFlowSpeed",40.78,40.78.......................................................... (mi/h)
"SegRunningTime",33.46,33.43................................................................. (s)
"SegRunningSpeed",36.67,36.71............................................................... (mi/h)
"SegThruDelay",20.369,20.043............................................................... (s/veh)
"SegTravelSpeed",22.8,22.95................................................................. (mi/h)
"SegThruStops",.596,.585................................................................ (stops/veh)
"SegSpatialStops",1.75,1.72.............................................................. (stops/mi)
"SegThruVolume",968.35,949.29.............................................................. (veh/h)
"SYSTEM"
"SystemTravelTime",107.3,107.3................................................................. (s)
"SystemTravelSpeed",22.87,22.87............................................................. (mi/h)
"SystemSpatialStops",1.73,1.73........................................................... (stops/mi)
"SystemBaseFreeFlowSpeed",40.78,40.78....................................................... (mi/h)
"NbrIntersections",3
"INTERSECTION:",1
"TimerPhaseAssign0",1,2,3,4,5,6,7,8
"Left Lane Group"
"TimerPhaseAssign",1,0,3,0,5,0,7,0.........................................................
"TimerGroupVolume",189.9,0,100,0,200,0,100,0............................................... (veh/h)
"TimerGroupUniformDelay",46.875,0,30.966,0,42.179,0,30.966,0.............................. (s/veh)
"TimerGroupIncDelay",2.534,0,.601,0,3.436,0,.601,0........................................ (s/veh)
"TimerGroupD3Delay",0,0,0,0,0,0,0,0........................................................ (s/veh)
"TimerGroupUniformStops",.952,0,.716,0,.851,0,.716,0................................... (stops/veh)
"TimerGroupIncStops",.03,0,.013,0,.04,0,.013,0......................................... (stops/veh)
"TimerGroupH3Stops",0,0,0,0,0,0,0,0.................................................... (stops/veh)
"TimerGroupFinalQue",0,0,0,0,0,0,0,0......................................................... (veh)
"Middle Lane Group"
"TimerPhaseAssign",0,2,0,4,0,6,0,8.........................................................
"TimerGroupVolume",0,1000,0,278.6,0,949.3,0,278.6.......................................... (veh/h)
"TimerGroupUniformDelay",0,12.58,0,38.907,0,18.971,0,38.907............................... (s/veh)
"TimerGroupIncDelay",0,1.469,0,1.876,0,1.072,0,1.876...................................... (s/veh)
"TimerGroupD3Delay",0,0,0,0,0,0,0,0........................................................ (s/veh)
"TimerGroupUniformStops",0,.358,0,.826,0,.566,0,.826................................... (stops/veh)
"TimerGroupIncStops",0,.025,0,.023,0,.019,0,.023....................................... (stops/veh)
"TimerGroupH3Stops",0,0,0,0,0,0,0,0.................................................... (stops/veh)
"TimerGroupFinalQue",0,0,0,0,0,0,0,0......................................................... (veh)
"Right Lane Group"
"TimerPhaseAssign",0,12,0,14,0,16,0,18.....................................................
"TimerGroupVolume",0,10,0,271.4,0,9.5,0,271.4.............................................. (veh/h)
"TimerGroupUniformDelay",0,13.86,0,38.95,0,13.684,0,38.95................................. (s/veh)
"TimerGroupIncDelay",0,.035,0,1.998,0,.027,0,1.998........................................ (s/veh)
"TimerGroupD3Delay",0,0,0,0,0,0,0,0........................................................ (s/veh)
"TimerGroupUniformStops",0,.415,0,.827,0,.408,0,.827................................... (stops/veh)
"TimerGroupIncStops",0,.025,0,.024,0,.02,0,.024........................................ (stops/veh)
"TimerGroupH3Stops",0,0,0,0,0,0,0,0.................................................... (stops/veh)
"TimerGroupFinalQue",0,0,0,0,0,0,0,0......................................................... (veh)
```

Figure B.23. Sample scenario output file content.

- INTERSECTION: The values in this section describe the performance of all movements at Intersection 1. Each value represents an average for the analysis period. This section is repeated for each numbered intersection.
- TimerPhaseAssign0: This variable lists the phase sequence using a dual-ring controller structure. The first four numbers indicate the sequence for Ring 1, as presented in the order listed. The last four numbers indicate the sequence for Ring 2. The numbers listed are the movement numbers associated with the through and left-turn movements.
- Left Lane Group: This text defines the section containing the data for the left-lane groups at the intersection. This lane group is used to describe the performance of any lane groups that serve a left-turn movement in one or more exclusive lanes. If there are no exclusive-lane left-turn lane groups,

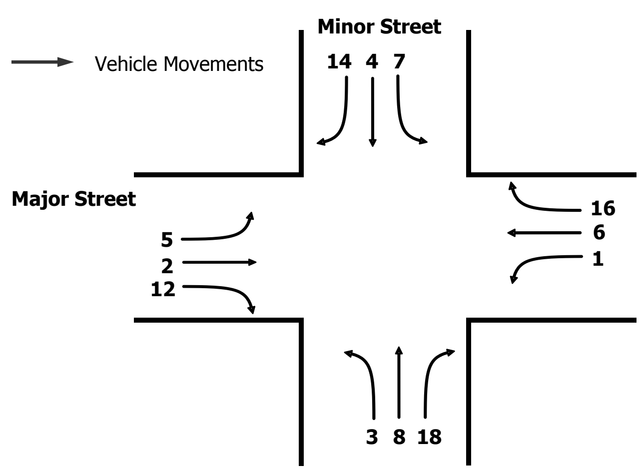


Figure B.24. Intersection traffic movements and numbering scheme.

then this lane group is used to describe the shared lane serving left-turn and through movements. This lane group is also used to describe the case for which there is only one lane on the approach.

- TimerPhaseAssign: This is a header row because it defines the order by which the values are listed in the subsequent rows for the specified lane group. For example, the row with the uniform delay (TimerGroupUniformDelay) shows 46.875 s/veh for Movement 1, 30.966 s/veh for Movement 3, and so forth.
- Middle Lane Group: This text defines the section containing the data for the middle-lane groups at the intersection. This lane group is used to describe the performance of any lane groups that serve a through movement in one or more exclusive lanes.
- Right Lane Group: This text defines the section containing the data for the right-lane groups at the intersection. This lane group is used to describe the performance of any lane groups that serve a right-turn movement in one or more exclusive lanes. If there are no exclusive-lane right-turn lane groups, then this lane group is used to describe the shared lane serving right-turn and through movements.

Software Installation

This section describes the steps involved in installing the reliability evaluation software. The installation process consists of establishing the folder structure for the software and saving the software in the appropriate folder. The installation process does not require administrative access to the computer. It does not make changes to the Windows registry, nor does it add any shortcuts to the desktop or program directory.

The steps for installing the reliability evaluation software are as follows:

Step 1. Create a folder for the HCM executable file (e.g., C:\Users\ksmith\Documents\USRE). Save the HCM executable file in this folder.

Step 2. Create a folder for the USCE workbook that is specific to the facility being evaluated (e.g., C:\Users\ksmith\Documents\USRE\10thStreet). Save the USCE workbook in this folder. Modify the workbook file name by attaching a suffix to indicate that it applies to the facility (e.g., C17_A06-4_G06-4_V2010_L08_ex.xls). In the context of evaluating various improvement strategies, this workbook would describe the comparator facility (typically, the comparator is the "existing" facility). Enter data in this workbook.

If one or more improvement strategies are being considered, copy the USCE comparator workbook and rename it to create a new workbook; one workbook should be created for each strategy being considered. Specifically, modify the workbook file name by attaching a suffix to indicate the specific strategy to which this workbook will apply (e.g., C17_A06-4_G06-4_V2010_L08_s1.xls).

Step 3. Create one folder for the USRE workbook that is specific to the facility being considered (e.g., C:\Users\ksmith\Documents\USRE\10thStreet\ex). Save the USRE workbook in this folder. Modify the workbook file name by attaching a suffix indicating the specific replication to which this workbook will apply (e.g., USRE-XY_ex-rep1.xls). Copy the USRE workbook and rename it to create a new workbook; create one workbook for each replication being considered.

If one or more improvement strategies are being considered, create one folder for each strategy being considered (e.g., C:\Users\ksmith\Documents\USRE\10thStreet\s1). Save a copy of the USRE workbook in this folder. Modify the workbook file name by attaching a suffix indicating the specific strategy and replication to which this workbook will apply (e.g., USRE-XY_s1-rep1.xls). Copy the USRE workbook and rename it; create one workbook for each replication being considered.

Step 4. Create one folder for the data files that is specific to the facility and replication identified in Step 3 (e.g., C:\Users\ksmith\Documents\USRE\10thStreet\ex\rep1). Repeat this step to create one folder for each replication (e.g., C:\Users\ksmith\Documents\USRE\10thStreet\ex\rep2, and so forth).

If one or more improvement strategies are being considered, create one folder for the data files that is specific to the facility, strategy, and replication identified in Step 3 (e.g., C:\Users\ksmith\Documents\ USRE\10thStreet\s1\rep1). Repeat this step to create one folder for each unique combination of strategy and replication (e.g., C:\Users\ksmith\Documents\USRE\10thStreet\s1\rep2).

Step 5. Use the USCE comparator workbook set up in Step 2 to create an HCM data set for the base conditions on the facility (i.e., no work zones, no special events). Save this base data set to each of the replication folders created in Step 4 for the comparator facility.

If a work zone, special event, or both occur during the RRP, then use the USCE workbook set up in Step 2 to create one HCM data set for each unique occurrence. Save this alternative data set to the same replication folders in which the base data set was saved. Each USCE workbook used to create an alternative data set may be renamed and saved, if desired (e.g., C17_A06-4_G06-4_V2010_L08_ex-wz1.xls).

If one or more improvement strategies are being considered, then the process outlined in the preceding paragraphs is repeated for each strategy. This process is described as follows:

a. For a given strategy, use the USCE workbook set up in Step 2 to create an HCM data set for the base conditions on the facility (i.e., no work zones, no special events). Save this base data set to each of the replication folders created in Step 4 for the specified strategy.
b. For a given strategy, if a work zone, special event, or both occur during the RRP, then use the USCE workbook set up in Step 2 to create an HCM data set for each unique occurrence. Save this alternative data set to the same replication folders in which the base data set was saved. Each USCE workbook used to create an alternative data set may be renamed and saved, if desired (e.g., C17_A06-4_G06-4_V2010_L08_s1-wz1.xls).

Step 6. Run the USRE comparator workbooks set up in Step 3. One workbook was set up for each replication. Each workbook should be coded so that the results from each run are saved in the appropriate replication folder. After each run, the appropriate replication folder should contain one scenario data set file and one scenario output file for each analysis period associated with that replication.

If one or more improvement strategies are being considered, then the process outlined in the preceding paragraph is repeated for each strategy. This process is described as follows: for a given strategy, run the USRE workbooks set up in Step 3, in which one workbook was set up for each replication. Each workbook should be coded so that the results from each run will be saved in the appropriate replication folder. After each run, the appropriate replication folder should contain one scenario data set file and one scenario output file for each analysis period associated with that replication.

Step 7. Use the Analysis worksheet in the USRE workbook to evaluate the results from the set of replications for each alternative.

Step 8. At the conclusion of the evaluation, archive (or delete) the files associated with this evaluation. As a minimum, the data files in the replication folders should be archived (or deleted) due to their large number.

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APPENDIX C

Recurring Demand for Freeway Scenario Generator

This appendix discusses freeway demand estimation for the scenario generators.

Demand Variability

Categorization of demand is done by defining demand patterns in the reliability reporting period (RRP). Specific days with similar demand level are put into one demand pattern. The basis of defining demand pattern consists of two dimensions that account for the monthly and weekly variability of demand in the RRP. Monthly variability usually highlights seasonal demand effect, and the weekly dimension shows the effect of daily variation in demand levels.

Demand level should be studied for the facility where the reliability analysis is performed. As one of the requirements, agencies or analysts should compile demand multipliers for each weekday for all months in the RRP. These demand multipliers give the ratio of demand for a day-month combination to the annual average daily traffic (AADT) and are used to generate demand values for later FREEVAL-RL (FREeway EVALuation–Reliability) runs. In the absence of facility-specific demand multipliers or local multipliers from nearby automated traffic recorders, the freeway scenario generator (FSG) can use its own embedded urban or rural default values. Table C.1 shows the demand multipliers for the I-40 eastbound (EB) case study. The text colors in Appendix C tables reflect the collection of patterns. The shading of cells provides conditional formatting on a green-to-red color scale, with lower-demand multipliers given a green shading and higher-demand multipliers, a red shading.

Demand patterns are defined on the basis of the demand multiplier distribution across the months and weekdays. This task is done by the analyst, although the user can select the FSG default demand pattern. For example, the demand pattern for I-40 EB case study was found to be seasonal across the monthly dimension; demand on Monday, Tuesday, and Wednesday fit in one group, and Thursdays and Fridays were in two additional separate groups as shown in Figure C.1. The demand pattern definition for I-40 EB is based on demand level comparison and categorization of days of week and months of year based on the demand level shown in Figure C.1. Table C.2 shows the demand pattern configuration across weekdays and months for the I-40 EB case study.

To estimate the probability of each demand pattern, RRP duration (in minutes) with a certain demand pattern is divided by the total RRP duration. Table C.3 presents a schematic of FSG demand patterns configuration for the I-40 EB case study. The demand pattern number is simply an indicator of each level of demand. It begins on the first day of the calendar, and a demand pattern number is assigned to each day inside the RRP.

Define $p_{\mathrm{DP}}(Z)$ as the probability of Demand Pattern Z, which is computed using Equation C.1:

$$p_{\mathrm{DP}}(Z) = \frac{\begin{array}{c}\text{Sum of SP minutes within}\\ \text{Demand Pattern } Z\end{array}}{\text{Sum of SP minutes in RRP}} \qquad \text{(C.1)}$$

where SP is study period.

For example, the probability of occurrence of Demand Pattern 5 at any time in the RRP is shown in Equation C.2:

$$p_{\mathrm{DP}}(5) = \frac{13 \times 6 \times 60}{261 \times 6 \times 60} = 4.98\% \qquad \text{(C.2)}$$

where the number of SPs (or days) with Demand Pattern 5 is 13, SP is equal to 6 h, and the total number of SPs in the RRP (or days in analysis) is 261.

There are two approaches in the FSG to generate the demand data required for FREEVAL: demand data poor and demand data rich.

Demand from AADT

When agencies do not have access to detailed demand information for a freeway facility, demand information is

(text continues on page 172)

Table C.1. Demand Multipliers for I-40 EB Case Study

Month	Day of Week				
	Monday	Tuesday	Wednesday	Thursday	Friday
January	0.996623	1.027775	1.040394	1.052601	1.081612
February	0.939253	1.010728	1.039214	1.092029	1.140072
March	1.043305	1.069335	1.063524	1.110921	1.171121
April	1.073578	1.087455	1.098238	1.161974	1.215002
May	1.076331	1.106182	1.113955	1.157717	1.210434
June	1.078043	1.085853	1.067470	1.138720	1.180327
July	1.082580	1.070993	1.102512	1.147279	1.184981
August	1.046045	1.052146	1.060371	1.093243	1.164901
September	1.016023	1.024051	1.023625	1.074782	1.152946
October	1.048981	1.045723	1.066986	1.107044	1.160954
November	0.974044	0.999947	1.041211	1.081541	1.070354
December	0.974785	0.956475	0.987019	0.916107	1.007695

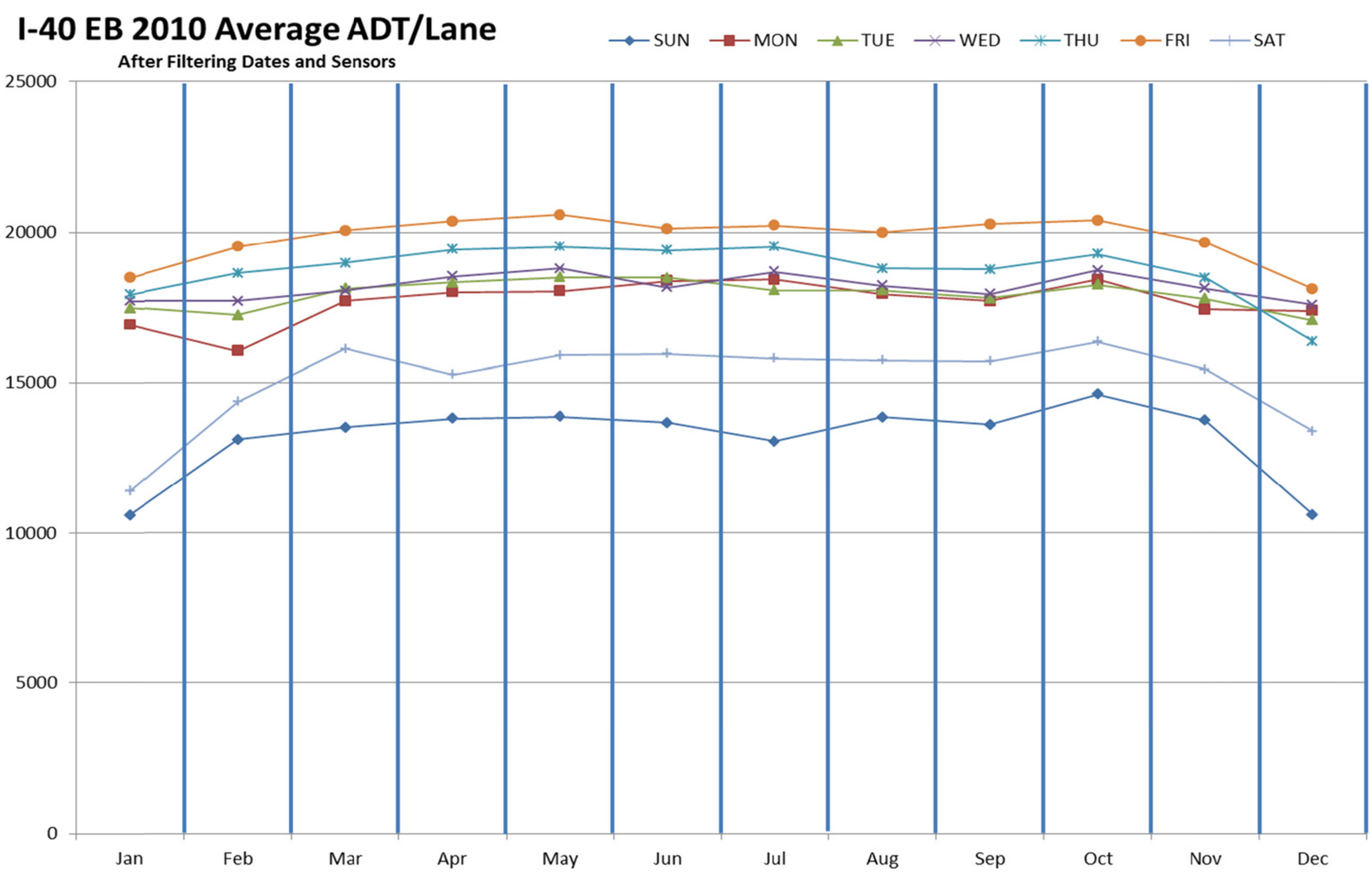


Figure C.1. Facility average ADT (average daily traffic) per lane by month and day of the week.

Table C.2. Demand Pattern Configuration for I-40 EB Case Study

	Monday	Tuesday	Wednesday	Thursday	Friday
January	1	1	1	2	3
February	1	1	1	2	3
March	4	4	4	5	6
April	4	4	4	5	6
May	4	4	4	5	6
June	7	7	7	8	9
July	7	7	7	8	9
August	7	7	7	8	9
September	10	11	11	12	12
October	10	11	11	12	12
November	10	11	11	12	12
December	1	1	1	2	3

Table C.3. Partial Listing of Demand Patterns Associated with I-40 EB Case Study

Week #	January	Monday	Tuesday	Wednesday	Thursday	Friday
Week 1	January	na	na	na	na	1/1/2010 (3)
Week 2	January	1/4/2010 (1)	1/5/2010 (1)	1/6/2010 (1)	1/7/2010 (2)	1/8/2010 (3)
Week 3	January	1/11/2010 (1)	1/12/2010 (1)	1/13/2010 (1)	1/14/2010 (2)	1/15/2010 (3)
Week 4	January	1/18/2010 (1)	1/19/2010 (1)	1/20/2010 (1)	1/21/2010 (2)	1/22/2010 (3)
Week 5	January	1/25/2010 (1)	1/26/2010 (1)	1/27/2010 (1)	1/28/2010 (2)	1/29/2010 (3)
Week 6	February	2/1/2010 (1)	2/2/2010 (1)	2/3/2010 (1)	2/4/2010 (2)	2/5/2010 (3)
Week 7	February	2/8/2010 (1)	2/9/2010 (1)	2/10/2010 (1)	2/11/2010 (2)	2/12/2010 (3)
Week 8	February	2/15/2010 (1)	2/16/2010 (1)	2/17/2010 (1)	2/18/2010 (2)	2/19/2010 (3)
Week 9	February	2/22/2010 (1)	2/23/2010 (1)	2/24/2010 (1)	2/25/2010 (2)	2/26/2010 (3)
Week 10	March	3/1/2010 (4)	3/2/2010 (4)	3/3/2010 (4)	3/4/2010 (5)	3/5/2010 (6)
Week 11	March	3/8/2010 (4)	3/9/2010 (4)	3/10/2010 (4)	3/11/2010 (5)	3/12/2010 (6)
Week 12	March	3/15/2010 (4)	3/16/2010 (4)	3/17/2010 (4)	3/18/2010 (5)	3/19/2010 (6)
Week 13	March	3/22/2010 (4)	3/23/2010 (4)	3/24/2010 (4)	3/25/2010 (5)	3/26/2010 (6)
Week 14	April	3/29/2010 (4)	3/30/2010 (4)	3/31/2010 (4)	4/1/2010 (5)	4/2/2010 (6)
Week 15	April	4/5/2010 (4)	4/6/2010 (4)	4/7/2010 (4)	4/8/2010 (5)	4/9/2010 (6)
Week 16	April	4/12/2010 (4)	4/13/2010 (4)	4/14/2010 (4)	4/15/2010 (5)	4/16/2010 (6)
Week 17	April	4/19/2010 (4)	4/20/2010 (4)	4/21/2010 (4)	4/22/2010 (5)	4/23/2010 (6)
Week 18	May	4/26/2010 (4)	4/27/2010 (4)	4/28/2010 (4)	4/29/2010 (5)	4/30/2010 (6)
Week 19	May	5/3/2010 (4)	5/4/2010 (4)	5/5/2010 (4)	5/6/2010 (5)	5/7/2010 (6)
Week 20	May	5/10/2010 (4)	5/11/2010 (4)	5/12/2010 (4)	5/13/2010 (5)	5/14/2010 (6)
Week 21	May	5/17/2010 (4)	5/18/2010 (4)	5/19/2010 (4)	5/20/2010 (5)	5/21/2010 (6)
Week 22	May	5/24/2010 (4)	5/25/2010 (4)	5/26/2010 (4)	5/27/2010 (5)	5/28/2010 (6)
Week 23	June	5/31/2010 (4)	6/1/2010 (7)	6/2/2010 (7)	6/3/2010 (8)	6/4/2010 (9)
Week 24	June	6/7/2010 (7)	6/8/2010 (7)	6/9/2010 (7)	6/10/2010 (8)	6/11/2010 (9)

Note: N/A = not applicable.

computed based on the AADT estimated for the facility along with hourly and daily demand multipliers. Mainline and ramp AADTs are entered in the Facility-Basics worksheet of FSG. Each detailed scenario is associated with a base scenario. Each base scenario is a combination of a demand pattern, weather, and incident event. The demand associated with each base scenario comes from the demand pattern for which that base scenario is generated. Thus, an aggregated demand multiplier for each demand pattern should be computed and applied for each detailed scenario to adjust the demand level.

The hourly variation should be incorporated for generating demand distributions for different 15-min time periods for FREEVAL-RL. Hourly demand distributions are entered in the Demand Hourly worksheet in FSG. Because FREEVAL-RL requires a 15-min demand distribution, 15-min demand distributions are estimated using linear interpolation. Define $K_t^{15\text{ minute}}$ as the portion of demand in the 15-min time period t, and $(D_i^t)_k$ as the hourly demand in segment i, time period t for detailed scenario k.

Equation C.3 shows how $(D_i^t)_k$ is computed. DMDP_k is the aggregated demand multiplier for scenario k across its defined demand pattern. This aggregation is done based on the number of days that the demand pattern has.

$$(D_i^t)_k = (4 \times K_t^{15\text{ minute}}) \times (\text{DMDP}_k) \times \left(\frac{\text{DAADT}_i}{24}\right) \qquad \text{(C.3)}$$

where DAADT_i is directional AADT on segment i.

Table C.4 presents the demand multipliers for each time period of detailed scenarios. Note that since the I-40 EB case study uses the data-rich approach, data presented in Table C.4 are just for illustrative purposes.

Demand from Sensor Data

In a data-rich environment, FSG has hourly demands for all time periods of an SP. A 15-min variation is already incorporated in the seed file. The only adjustment that needs to be inserted for generating the demand for a detailed scenario is the daily demand multiplier for the seed SP, which is denoted by DM_{Seed}. The hourly demand in segment i, time period t for detailed scenario k is then computed using Equation C.4:

$$(D_i^t)_k = \left(\frac{(D_i^t)_{\text{Seed}}}{\text{DM}_{\text{Seed}}}\right)(\text{DMDP}_k) \qquad \text{(C.4)}$$

In a data-rich approach, what passes to FREEVAL-RL is basically $\left(\frac{\text{DMDP}_k}{\text{DM}_{\text{Seed}}}\right)$. Table C.5 shows the demand multipliers for the I-40 EB case study scenario Number 2117.

Demand Example: I-40 Study Site

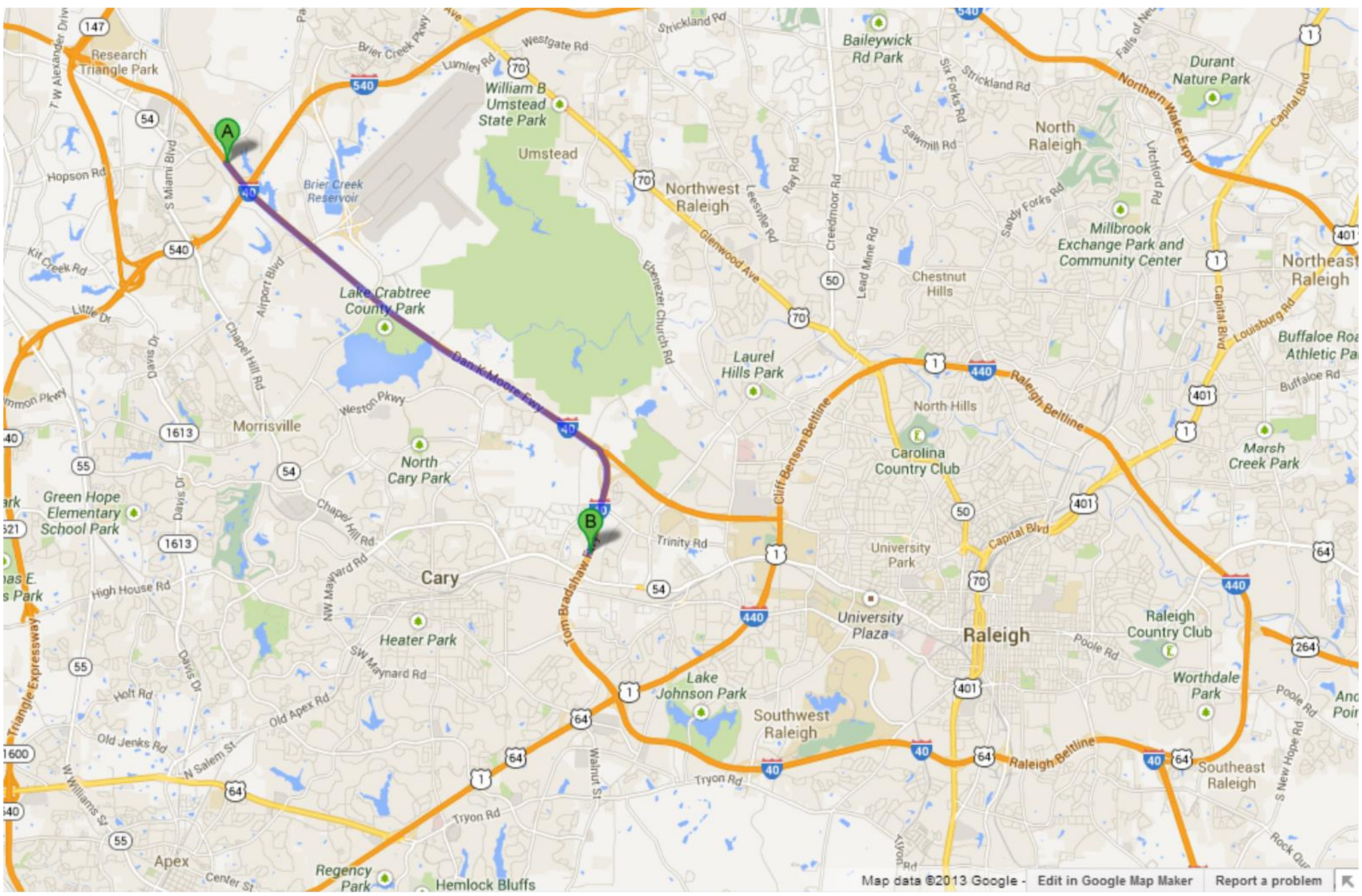
The freeways methodology was applied to a 12.5-mi freeway facility on I-40 EB between Mile Markers 278.5 and 291 near Raleigh, North Carolina. The case study facility has a speed limit of 65 mph and a free-flow speed of 70 mph. The RRP over which the analysis was carried included all weekdays of calendar year 2010 in a study period from 2:00 to 8:00 p.m. Figure C.2 shows the location of the study site. The facility is primarily a commuter route that connects Durham, North Carolina (Point A) to Raleigh (Point B) and passes through Research Triangle Park, a major employment center in the area. The two-way facility AADT was approximately 120,000 in 2010; the EB facility experiences recurring congestion in the p.m. peak period.

Table C.4. Minute Demand Adjustment Factors

Time Period	DM	Time Period	DM
t= (12:00 AM to 12:00 AM)	1.29	t= (12:00 AM to 12:00 AM)	1.40
t= (12:00 AM to 12:00 AM)	1.33	t= (12:00 AM to 12:00 AM)	1.31
t= (12:00 AM to 12:00 AM)	1.36	t= (12:00 AM to 12:00 AM)	1.22
t= (12:00 AM to 12:00 AM)	1.36	t= (12:00 AM to 12:00 AM)	1.22
t= (12:00 AM to 12:00 AM)	1.44	t= (12:00 AM to 12:00 AM)	1.04
t= (12:00 AM to 12:00 AM)	1.53	t= (12:00 AM to 12:00 AM)	0.98
t= (12:00 AM to 12:00 AM)	1.62	t= (12:00 AM to 12:00 AM)	0.91
t= (12:00 AM to 12:00 AM)	1.62	t= (12:00 AM to 12:00 AM)	0.80
t= (12:00 AM to 12:00 AM)	1.80	t= (12:00 AM to 12:00 AM)	0.78
t= (12:00 AM to 12:00 AM)	1.70	t= (12:00 AM to 12:00 AM)	0.77
t= (12:00 AM to 12:00 AM)	1.60	t= (12:00 AM to 12:00 AM)	0.76
t= (12:00 AM to 12:00 AM)	1.60	t= (12:00 AM to 12:00 AM)	0.80

Table C.5. Demand Multipliers for I-40 EB Detailed Scenario Number 2117

Time Period	DM	Time Period	DM
t= (12:00 AM to 12:00 AM)	1.07	t= (12:00 AM to 12:00 AM)	1.07
t= (12:00 AM to 12:00 AM)	1.07	t= (12:00 AM to 12:00 AM)	1.07
t= (12:00 AM to 12:00 AM)	1.07	t= (12:00 AM to 12:00 AM)	1.07
t= (12:00 AM to 12:00 AM)	1.07	t= (12:00 AM to 12:00 AM)	1.07
t= (12:00 AM to 12:00 AM)	1.07	t= (12:00 AM to 12:00 AM)	1.07
t= (12:00 AM to 12:00 AM)	1.07	t= (12:00 AM to 12:00 AM)	1.07
t= (12:00 AM to 12:00 AM)	1.07	t= (12:00 AM to 12:00 AM)	1.07
t= (12:00 AM to 12:00 AM)	1.07	t= (12:00 AM to 12:00 AM)	1.07
t= (12:00 AM to 12:00 AM)	1.07	t= (12:00 AM to 12:00 AM)	1.07
t= (12:00 AM to 12:00 AM)	1.07	t= (12:00 AM to 12:00 AM)	1.07
t= (12:00 AM to 12:00 AM)	1.07	t= (12:00 AM to 12:00 AM)	1.07
t= (12:00 AM to 12:00 AM)	1.07	t= (12:00 AM to 12:00 AM)	1.07



Source: © 2013 Google.

Figure C.2. I-40 facility location.

Table C.6. Demand Factors: Ratio of ADT to AADT by Month and Day of Week

	SUN	MON	TUE	WED	THU	FRI	SAT
Jan	0.617609	0.999005	1.030232	1.042881	1.055117	1.084198	0.662407
Feb	0.763747	0.941499	1.013144	1.041699	1.094640	1.142797	0.837179
Mar	0.794913	1.045799	1.071891	1.066066	1.113577	1.173921	0.940873
Apr	0.817347	1.076144	1.090055	1.100863	1.164751	1.217906	0.911421
May	0.815670	1.078904	1.108827	1.116618	1.160484	1.213328	0.933496
Jun	0.805796	1.080620	1.088449	1.070022	1.141443	1.183148	0.942226
Jul	0.764001	1.085168	1.073553	1.105148	1.150022	1.187813	0.933042
Aug	0.801063	1.048545	1.054661	1.062905	1.095856	1.167686	0.911527
Sep	0.768024	1.018452	1.026499	1.026072	1.077352	1.155702	0.893950
Oct	0.825240	1.051489	1.048223	1.069537	1.109691	1.163729	0.924886
Nov	0.756585	0.976373	1.002337	1.043700	1.084126	1.072912	0.829501
Dec	0.586780	0.977116	0.958762	0.989379	0.918297	1.010103	0.744283

Traffic demand data were estimated from counts extracted from permanent side-fire radar sensors placed along the mainline of the facility, supplemented with temporary tube counters placed at the on- and off-ramps for a 2-week period (there are no permanent sensors on the ramps). Side-fire sensor data were collected for all of 2010 at the 15-min level, and daily per lane volumes were calculated at each sensor to determine combinations of days and months that operated similarly. Table C.6 shows the average daily traffic (ADT) per lane trends for 2010, when Mondays through Wednesdays experienced very similar demand levels. Thursday demand levels were more elevated, and Friday's were the highest. Although the seasonal variation was not as significant, four seasons, each encompassing 3 months (December to February; March to May; June to August; and September to November) were selected to group similar demand months, as well as months with similar weather conditions.

This process resulted in the identification of 12 demand groups, or patterns. Daily and monthly demand factors were calculated from the ratio of ADT for each combination of month and day for 2010 to the AADT, as shown in Table C.6. These values were then averaged for each of the 12 demand patterns emerging from the data. These patterns are depicted in Table C.6 for each collection of contiguous cells with the same cell color background and border color. In the following calibration, the overall demand levels are adjusted to determine the best demand level that recreates the observed operations.

APPENDIX D

Weather- and Incident-Related Crash Frequencies

This appendix provides an overview of the weather and incident data requirements needed to run a SHRP 2 Project L08 travel time reliability analysis. It also provides basic data collection and analysis guidance for data-rich agencies wishing to make their analyses more precise by including high-detail, facility-specific weather and incident statistics in their reliability analyses.

Travel time reliability depends heavily on weather and incident events, which must be properly taken into account in any prediction or analytical evaluation of reliability. Acquisition and Processing of Weather Data provides an overview of weather data sources (including databases compiled as part of the SHRP 2 L08 project) and a review of classifying typical weather data into *Highway Capacity Manual 2010* (HCM2010) (Transportation Research Board of the National Academies 2010) weather types. Acquisition and Processing of Incident Data provides guidance on converting incident logs into HCM incident types and expanding crash records to HCM-type incidents. A brief review of the default incident values included in the SHRP 2 Project L08 computational engines is also included.

Because the freeways and urban streets reliability methodologies employ different measures of incidents and weather, the data processing sections clearly differentiate between them. Table D.1 shows the basic requirements and optional inputs for both weather and incident data.

Weather

Urban Streets

The SHRP 2 Project L08 urban streets software tool does not allow for custom input of weather. The analyst must choose one of the 284 cities for which weather has been simulated.

Freeways

A database of 10-year average weather probabilities was compiled by the SHRP 2 Project L08 freeways team for 101 U.S. metropolitan areas. If the database does not contain the subject freeway's location or if the analyst prefers to use facility-specific weather, he or she can use this document to prepare and input weather data. Weather data sources and a methodology consistent with the one used to compile the database are presented below in Acquisition and Processing of Weather Data.

Incidents

Urban Streets

The SHRP 2 L08 project urban streets software tool limits user input to annual crash rates for segments and intersections of interest. This document provides guidance on calculating these rates from crash logs or national crash prediction methodologies, or both.

Freeways

The freeway facilities spreadsheet (FREEVAL-RL) gives the analyst more opportunities for customizing incident input. This document provides guidance on extracting or estimating the probability of incident occurrence by incident type by month of year. The calculation is based on the proportion of the analysis period duration that the freeway segment is subject to a given incident type.

For data-poor agencies, advice on how to expand crash logs into incidents, how to convert crash data into number of lanes closed, and how to customize average incident durations is provided.

FREEVAL-RL also provides the option to customize free-flow speed adjustments and capacity adjustment factors—which would modify the speed–flow curves used by the methodology—but these items are outside the scope of this data collection and processing guide.

Table D.1. Data Requirements

Methodology	Data Set	Data Poor	Data Rich
Urban Streets	Weather	None (database only).	None (database only)
	Incidents	Annual crash rate for each segment and intersection.	No additional options
Freeways	Weather	None (database available). Editing of average weather event duration is optional.	Probability of occurrence (duration based) for 11 HCM weather types, by month
			Free-flow speed adjustment factor
			Capacity adjustment factor
	Incidents	None (prediction methods available). Editing of incident type distribution and average incident durations is optional.	Probability of occurrence (duration based) for six incident types, by month
			Free-flow speed adjustment factor
			Capacity adjustment factor

Acquisition and Processing of Weather Data

Weather Data Sources

The ideal weather data set would include a year's worth of 15-min weather reports collected near the facility being studied. In the likely case that 15-min weather data are not available, hourly weather reports published by the Federal Highway Administration's (FHWA) CLARUS system, the National Oceanic and Atmospheric Administration (NOAA), Weather Underground, and others can be used.

The CLARUS system was developed by FHWA to compile and distribute real-time atmospheric information (Federal Highway Administration 2012). There are three ways to obtain weather data using the CLARUS system: (1) subscribing to a report that is periodically updated, (2) using latest quality-checked observations on the map interface, and (3) retrieving an on-demand report for weather observations. A major disadvantage of the CLARUS system is the relatively small number of stations, especially in the south and southeast regions (see Figure D.1).

NOAA monitors weather across the United States. For purposes of hourly weather data collection, their meteorological aviation reports are the most useful. They contain several data points that are of interest to HCM-type weather calculation, including temperature, visibility, wind speed, and precipitation.

Weather Underground's historical hourly weather reports—which can be downloaded freely in .csv format from www.wunderground.com—rely on the meteorological aviation reports to archive all these metrics for almost every town and city in the United States (see Table D.2). These reports were used by the SHRP 2 L08 project freeways methodology to develop 10-year averages of weather occurrence probabilities for 101 metropolitan areas in the United States. The L08 project FREEVAL-RL software tool contains this database and can be a valuable resource for data-poor agencies that do not wish to compile and analyze weather reports.

A similar database of 284 urban areas was assembled for use with the urban streets methodology. Unlike the freeway facilities tool, the urban streets spreadsheet does not allow for custom input of weather. In other words, the analyst is limited to the 284 urban areas included in the database. Instead of collecting and averaging hourly weather reports, the urban streets project team simulated hourly weather by using a Monte Carlo simulation based on monthly weather statistics collected from the National Climatic Data Center (2011).

Guidance for data-rich agencies that wish to collect, compare, or analyze facility-specific weather data is included below.

To be able to classify weather into HCM format, certain measurements must be part of the time-stamped weather reports. These items are illustrated in Figure D.2.

Weather Data Processing

SHRP 2 Project L08 Urban Street Type

The urban streets spreadsheet does not allow custom input of weather. The user is limited to the 284 cities contained in the spreadsheet. This section is thus included for sake of completeness and in case the analyst wishes to compare facility-specific weather with the freeway spreadsheet's weather. If so, the facility's weather data must be collected and classified into one of the following weather types:

- Rainfall;
- Snowfall;
- Wet pavement, no rain;
- Icy pavement, no snow; or
- Clear, dry weather.

If the weather reports include a column with weather condition (e.g., cloudy, rainy, windy), classifying weather can be

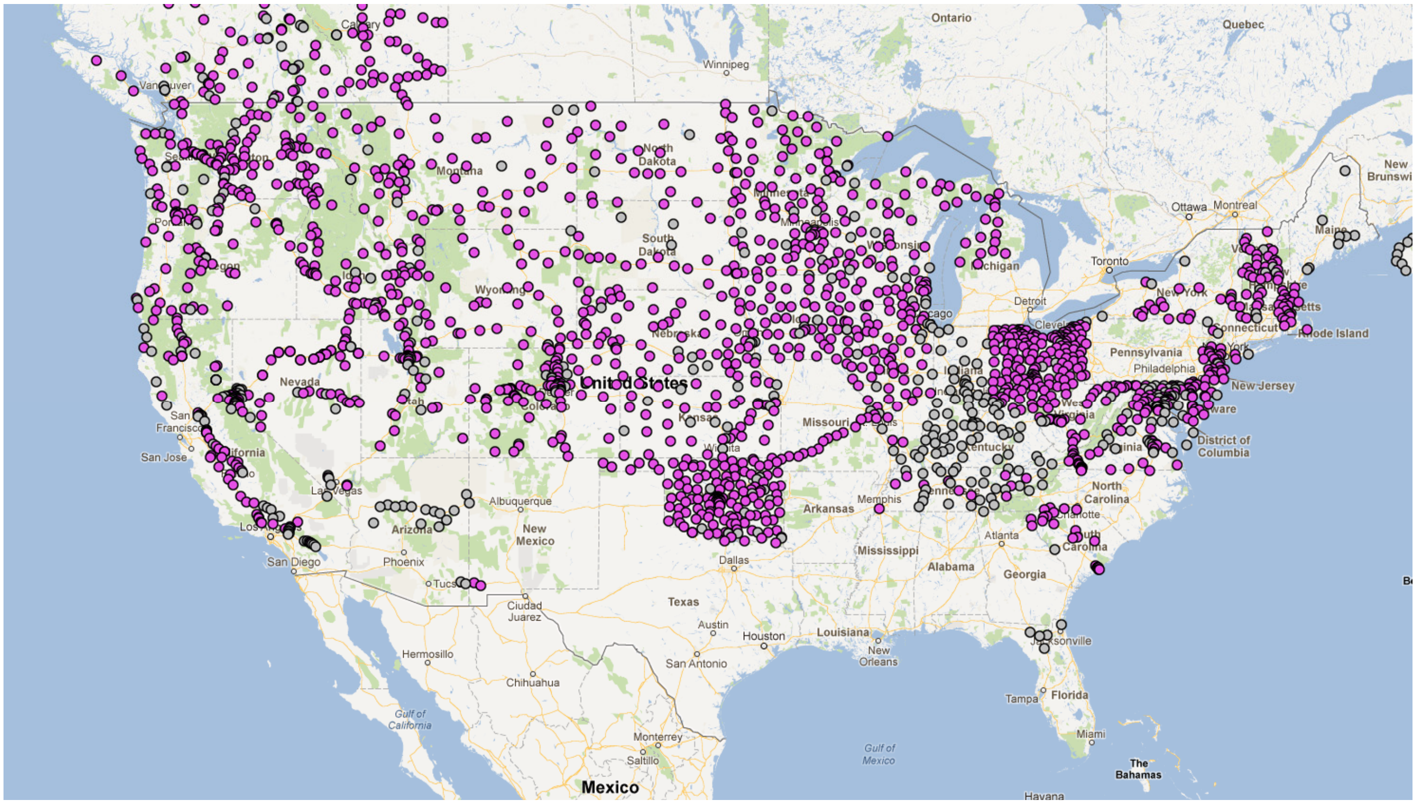


Figure D.1. Screenshot of CLARUS interactive weather data map.

Table D.2. Sample of Weather Underground Hourly Weather Reports

Hourly Observations

Time (PDT)	Temp.	Dew Point	Humidity	Pressure	Visibility	Wind Dir	Wind Speed	Gust Speed	Precip	Events	Conditions
12:53 AM	53.1 °F	45.0 °F	74%	30.09 in	10.0 mi	West	8.1 mph	-	N/A		Mostly Cloudy
1:53 AM	53.1 °F	45.0 °F	74%	30.06 in	10.0 mi	WSW	4.6 mph	-	N/A		Scattered Clouds
2:53 AM	53.1 °F	45.0 °F	74%	30.06 in	10.0 mi	WSW	6.9 mph	-	N/A		Scattered Clouds
3:53 AM	53.1 °F	45.0 °F	74%	30.05 in	10.0 mi	WSW	8.1 mph	-	N/A		Partly Cloudy
4:53 AM	53.1 °F	44.1 °F	71%	30.05 in	10.0 mi	West	8.1 mph	-	N/A		Partly Cloudy
5:53 AM	53.1 °F	45.0 °F	74%	30.04 in	10.0 mi	SW	6.9 mph	-	N/A		Overcast
6:53 AM	54.0 °F	45.0 °F	72%	30.04 in	10.0 mi	SW	8.1 mph	-	N/A		Overcast
7:53 AM	55.0 °F	46.0 °F	72%	30.03 in	10.0 mi	SE	10.4 mph	-	N/A		Overcast
8:53 AM	55.9 °F	46.0 °F	69%	30.03 in	10.0 mi	SSE	13.8 mph	-	N/A		Overcast
9:53 AM	55.9 °F	45.0 °F	67%	30.02 in	10.0 mi	SSE	12.7 mph	-	N/A		Overcast
10:53 AM	57.9 °F	46.0 °F	65%	30.01 in	10.0 mi	SSE	12.7 mph	-	N/A		Overcast
11:07 AM	59.0 °F	46.4 °F	63%	30.01 in	10.0 mi	SSE	12.7 mph	-	N/A		Overcast
11:41 AM	55.4 °F	50.0 °F	82%	30.03 in	2.5 mi	SSW	12.7 mph	-	0.00 in	Rain	Light Rain
11:53 AM	57.0 °F	51.1 °F	81%	30.03 in	2.0 mi	SSW	11.5 mph	-	0.01 in	Rain	Light Rain
12:06 PM	57.2 °F	51.8 °F	82%	30.02 in	2.0 mi	SSW	11.5 mph	-	0.01 in	Rain	Light Rain

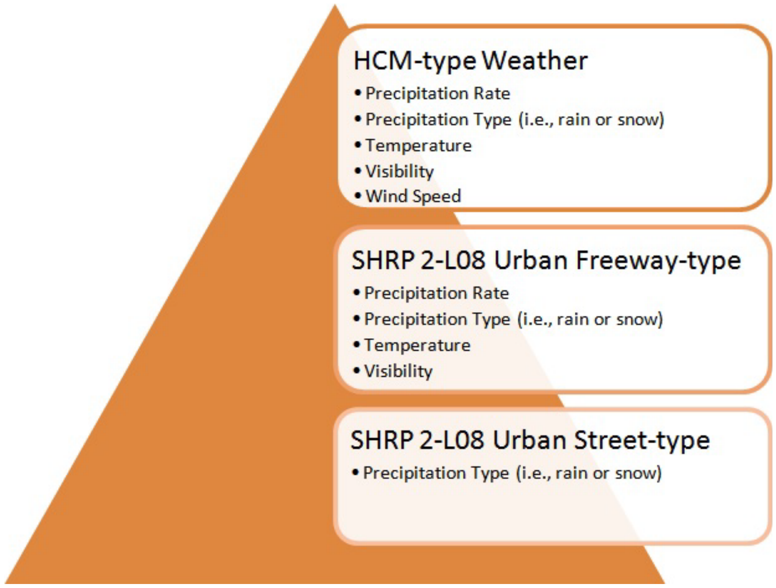


Figure D.2. Schematic of weather data quality.

as simple as setting filters and counting observations. If not, the precipitation rate column can be used to determine whether there was rainfall or snowfall. Often different conditions will need to be lumped into a single category. For example, thunderstorm, light rain, or scattered showers would need to be categorized together as rain.

In the likely case that pavement condition is not part of the weather report, it can be assumed that pavement remains wet or icy for 1 h after a rain or snow event. If 15-min weather data are available, these drying estimates can be replaced with 30 min.

After classifying the weather observations, it is possible to compute the probabilities of weather occurrence for each weather type. In 1 year, there should be 8,760 (365 × 24) hourly observations. The probability of occurrence of a weather type is simply the ratio of the number of observations of that particular weather type in 1 year to 8,760. Table D.3 shows the classification of a year's hourly observations for Springfield, Illinois.

Table D.3. Sample Output for Comparison with SHRP 2 Project L08 Urban Streets Weather (Springfield, Illinois)

Type	No. of Hours	Percentage (%)
Clear, dry	7,837	89.46
Rainfall	470	5.37
Snowfall	210	2.40
Wet pavement	194	2.21
Icy pavement	49	0.56

SHRP 2 Project L08 Freeway Type

Unlike the urban streets spreadsheet, the SHRP 2 Project L08 freeways spreadsheet does allow for custom input of weather (see Table D.4). For this purpose, weather data must be collected and classified into one of the following weather types:

- Medium rain;
- Heavy rain;
- Light snow;
- Light-medium snow;
- Medium-heavy snow;
- Severe cold;
- Low visibility;
- Very low visibility;
- Minimal visibility; or
- Nonsevere weather.

These weather types were adopted from those in the HCM capacity adjustment table. A few weather types with negligible capacity reductions (i.e., high and very high wind, light rain, and cold and very cold temperature) were omitted to decrease the computational complexity of the scenario generator. If these weather types are encountered, they should be counted as nonsevere weather.

Classifying weather reports into these categories is slightly more time-consuming than in the Project L08 urban streets method, but it can still be done easily with the use of a spreadsheet. Data columns that include the precipitation type (i.e., snow or rain), precipitation rate (in inches per hour), temperature, and visibility (in miles) should be used in conjunction with the thresholds in Table D.4 to classify each weather report row. In cases for which two or more weather types

Table D.4. SHRP 2 Project L08 Freeways Weather Input

	Weather Categories (based on HCM 2010 Chapter 10: Freeway Facilities)										
Month	**Med Rain**	**Heavy Rain**	**Light Snow**	**LM Snow**	**MH Snow**	**Heavy Snow**	**Severe Cold**	**Low Vis**	**Very Low Vis**	**Min Vis**	**Normal Weather**
January	0.724%	0.580%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	98.6963%
February	1.644%	1.001%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	97.3545%
March	0.727%	0.117%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	99.1559%
April	0.757%	0.112%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	99.1311%
May	0.175%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	99.8249%
June	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	100.0000%
July	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	100.0000%
August	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	100.0000%
September	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	100.0000%
October	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	100.0000%
November	0.806%	0.840%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	98.3542%
December	2.473%	1.010%	0.000%	0.000%	0.000%	0.000%	0.000%	0.175%	0.000%	0.000%	96.3428%
Average Duration for Weather Type(min):	46.2	30.4	33.3	19.0	17.0	0.0	0.0	36.4	0.0	76	
Default Capacity Adjustment Factor:	92.76%	85.87%	95.71%	91.34%	88.96%	77.57%	91.55%	90.33%	88.33%	89.51%	100.00%
Default FFS Adjustment Factor:	94.00%	93.00%	89.00%	88.00%	86.00%	85.00%	94.00%	94.00%	93.00%	93.00%	100.00%

apply (e.g., severe cold with light snow), the analyst should choose the weather type with the highest capacity reduction.

After classifying the weather observations, it is possible to compute the probabilities of weather occurrence for each weather type. In 1 year, there should be 8,760 (365 × 24) hourly observations. The probability of occurrence of a weather type is simply the ratio of the number of hourly observations of that weather type to 8,760. Table D.5 shows an example from the HCM2010 of capacity adjustment factors for weather conditions in Iowa.

HCM Type

This section is included here for sake of completeness only, since none of the reliability methodologies use the full breadth of the HCM weather capacity adjustments (for an example, see Table D.5). Should the analyst wish to conduct a thorough accounting of weather, he or she should not omit any weather type. In other words, Project L08 freeway weather should be augmented to include the following:

- Light rain;
- Cold temperature;
- Very cold temperature;
- High wind; and
- Very high wind.

Adding these weather types would increase the number of possible classifications from 11 to 16.

Table D.5. CAFs for Weather Conditions

Type of Condition	Intensity of Condition	Percent Reduction in Capacity: Average	Percent Reduction in Capacity: Range
Rain	>0 ≤ 0.10 in./h	2.01	1.17–3.43
	>0.10 ≤ 0.25 in./h	7.24	5.67–10.10
	>0.25 in./h	14.13	10.72–17.67
Snow	>0 ≤ 0.05 in./h	4.29	3.44–5.51
	>0.05 ≤ 0.10 in./h	8.66	5.48–11.53
	>0.10 ≤ 0.50 in./h	11.04	7.45–13.35
	>0.50 in./h	22.43	19.53–27.82
Temperature	<50°F ≥ 34°F	1.07	1.06–1.08
	<34°F ≥ –4°F	1.50	1.48–1.52
	<–4°F	8.45	6.62–10.27
Wind	>10 ≤ 20 mi/h	1.07	0.73–1.41
	>20 mi/h	1.47	0.74–2.19
Visibility	<1 ≥ 0.50 mi	9.67	One site
	<0.50 ≤ 0.25 mi	11.67	One site
	<0.25 mi	10.49	One site

Exhibit 10-15
Capacity Reductions due to Weather and Environmental Conditions in Iowa

Source: *Highway Capacity Manual 2010* (Transportation Research Board of the National Academies 2010).

Acquisition and Processing of Incident Data

Both SHRP 2 Project L08 computation engine spreadsheets include default values for incident statistics as part of their methodology, but their approach differs. The urban streets methodology simply asks for an annual crash rate and uses hard-coded default values, but the freeways methodology allows the user to substitute defaults with facility-specific values. This section provides guidance on the input of basic incident or crash rates and the calculation of facility-specific values.

Incident Data Source

The ideal outcome of the incident data collection effort is an annual incident rate and a table with the percentage frequency and average duration of incidents categorized by

- Type (e.g., accident, breakdown, debris);
- Severity (e.g., property damage only, injuries);
- Lane closure effect (e.g., shoulder closed, one lane closed, two lanes closed); and
- Location (i.e., intersection or segment).

This ideal outcome can only be achieved with high-detail, incident-by-incident logs. For the purposes of this research, agencies across the United States were contacted and suitable logs were obtained on a case-by-case basis. Because of the complexity involved in obtaining these data, the L08 project freeways methodology includes default incident probabilities and prediction methodologies for estimating crash rates. The urban streets methodology simply requires annual crash rates. Figure D.3 provides a schematic of incident data quality.

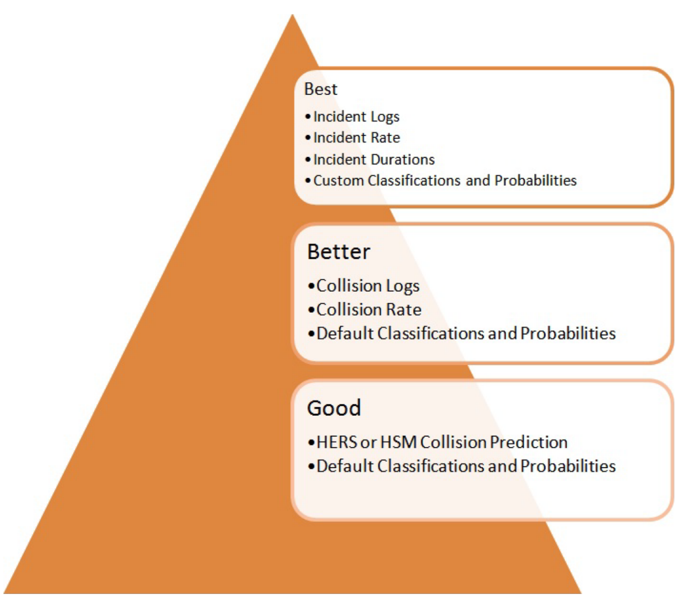


Figure D.3. Schematic of incident data quality.

For customization of a Project L08 reliability analysis, different incident methodologies can be used depending on the incident data quality illustrated in Figure D.3. In general, each agency is considered either data rich or data poor. Data-rich agencies are those agencies with an active traffic management center that monitors and archives incident data for freeways and arterials on a daily basis. Data-poor agencies are those agencies without traffic management center operations or those with no access to incident archives. In the absence of incident-log data, those agencies will require a methodology for estimating the number of incidents and default values, which is provided later in this guide.

Collision records, which only contain crashes, are a more easily obtainable source of data. Most state departments of transportation and highway patrol agencies collect crash data on public roadway facilities. However, most agencies only publish monthly or yearly summaries of collisions, which are appropriate for deriving a yearly crash rate but not for classifying collisions by severity type, location, and lane closure effect. Examples of these systems include California's SWITRS (California Highway Patrol) and Kentucky's Collision Analysis for the Public (Kentucky State Police).

If collision data cannot be obtained, it is possible to use one of the crash prediction methodologies developed by FHWA's Highway Economic Requirements System (HERS) (Federal Highway Administration 2005) or the *Highway Safety Manual* (HSM) (American Association of State Highway and Transportation Officials 2010). Both methodologies predict the collision rate based on the roadway's geometry, number of access points, daily volumes, and so forth. The freeways software tool (FREEVAL-RL) has the ability to run HERS prediction methodology on the input geometries and volumes.

Through techniques explained in the following sections, collision data can then be expanded to incident data. The SHRP 2 Project L08 freeways spreadsheet allows input of both crash-only and incident data. If crash data are entered, the spreadsheet expands it by using a national default of 4.9 incidents per crash or a user-input ratio. Expanded crash data can be further categorized into incident types, converted into number of lanes closed, and categorized into crash severity type.

Incident Data Requirements

Depending on the evaluation type (e.g., existing or future facilities, urban streets or urban freeways), data-rich agencies and data-poor agencies should follow different procedures and methodologies to estimate and identify incident occurrence probabilities and incident duration by incident type and lane closure type. Default values, which were compiled from incident data sets from various geographical locations, are provided in case of lack of data.

SHRP 2 Project L08 Urban Street Type

Similar to its treatment of weather events, the SHRP 2 L08 project methodology has a simple, user-friendly approach to crash data input. In this case, the only requirement is yearly crash frequency by segment. In case the analyst wishes to account for higher or lower crash rates during work zones and events, fields for adjustment factors are provided. Table D.6 shows the urban streets crash data input screen.

Agencies with "best" or "better" incident data quality will be able to count the number of crashes in a year for each segment and input the rates into the Project L08 urban streets methodology spreadsheet (see Incident Data Processing below). Agencies with "good" incident data quality may use the urban arterials HSM method or the HERS arterial crash prediction (see the urban arterials portion of Table D.6). If the HERS arterial crash prediction method is chosen, the analyst must convert from crashes per 1,000,000 vehicle miles traveled (1MVMT) to crashes per year by using the segment length and annual average daily traffic (AADT). If better data are lacking, the approximate national proportion of crashes occurring at intersections (40%) (Federal Highway Administration 2009) should be used to separate the rates into the segment and intersection columns. The annual crash rate can be computed by using Equation D.1:

$$\text{Annual crash rate} = \left(\begin{matrix}\text{rate per MVMT} * \text{segment length} \\ * \text{AADT} * 365\end{matrix}\right) \Big/ 10^6 \tag{D.1}$$

The output of the urban arterials HSM method is already in crashes per year and thus requires no conversion.

SHRP 2 Project L08 Freeway Type

The SHRP 2 Project L08 freeways methodology lets the user choose among different levels of data quality. Option A—for data-poor facilities—prompts the user either to run the HERS model or directly input crash or incident rates. If the agency wishes to input custom rates, it should follow the instructions under Incident Data Processing below and convert them from crashes per month or incidents per month to crashes per 100 MVMT (100MVMT) or incidents per 100MVMT.

This conversion, which can be calculated using Equation D.2, will require knowing the segment length in miles (L) and the AADT where the crashes or incidents occurred. Number of days corresponds to the time span of the crash or incident data set.

$$\text{Rate per 100MVMT} = \frac{\text{Number of crashes or incidents} * 10^8}{(\text{AADT} * L * \text{Number of days})} \tag{D.2}$$

Option A also gives data-poor agencies the option of specifying a facility-specific crashes-to-incident conversion factor (the national default is 4.9) and custom annual incident type distribution and durations (see Table D.7).

Option B—for data-rich agencies—expands on these items and allows the user to enter month-by-month incident occurrence probabilities for each incident type (see Incident Data Processing below).

Furthermore, if facility-specific free-flow speed adjustment factors and capacity adjustment factors are known, they can be entered in place of the HCM defaults. These values will modify the speed–flow curves used by the methodology. These adjustments will not be found in incident logs and are thus outside the scope of this guide.

Incident Data Processing

While the previous sections describe sources of incident data and minimum data quality requirements, this section focuses on data processing. The following sections provide guidance on preparing incident data for entry into the SHRP 2 Project L08

Table D.6. SHRP 2 Project L08 Urban Streets, Crash Data Input Screen

Crash Data ↖ see note					
Segment Number	Segment Boundary Intersections	Crash Frequency cr/year		Intersection Number	Crash Frequency, cr/year
				1	65
1	1 to 2	15		2	66
2	2 to 3	16		3	67
3	3 to 4			4	
4	4 to 5			5	
5	5 to 6			6	
6	6 to 7			7	
7	7 to 8			8	
8	8 to 9			9	

Work Zone and Special Event Crash Frequency Adjustment Factors ↖ see note		
	Segment	Intersection
Alternative 1	1.1	1.2
Alternative 2		
Alternative 3		

Table D.7. SHRP 2 Project L08 Freeways, Incident Data Input Screen

Option A: for Data Poor Facilities:

Step 1: Enter Incident or Crash Rate (Per 100 million Vehicle Mile):

Entry Data are : ○ Incident Rates ◉ Crash Rates

IF Monthly Crash Rates will be Entered Below Then Enter Site Specific Crash to Incident Rate Ratio	7	National Default Ratio is 4.9

User can enter the rates, or use HERS model.

Month	Rate
January	164.5
February	164.5
March	164.5
April	164.5
May	164.5
June	164.5
July	164.5
August	164.5
September	164.5
October	164.5
November	164.5
December	164.5

If local crash rates are not availble, Press "Calculate Crash Rate based on HERS Model" to estimate crash rate from National Defaults

Calculate Crash Rate Based on HERS Model

Urban Streets Computational Engine and Freeways Computational Engine.

Figure D.4 represents the overall process for both existing and future facility evaluations and for both data-rich and data-poor agencies. A delineation can be made between an existing evaluation—which is designed for a "before" or opening day analysis stage—and a future evaluation, which is more appropriate for a long-term analysis stage.

For an existing facilities evaluation, data-rich agencies have the option to convert their incident-log data to a compatible format for input in the Project L08 analysis procedures. Data-poor agencies typically lack local incident data but have access to local crash data, which are identified as one of the common incident types. These agencies can populate incident frequency based on crash data. A procedure to estimate incidents from crash data is provided later in this guide.

For planned or future facilities, both data-rich and data-poor agencies would have to perform extra steps in order to estimate facility-specific incident frequency. When sufficient traffic and geometry information is available, crash frequency for arterials can be estimated using the crash prediction procedures available in the HSM. Alternatively, in a situation in which only planning-level parameters (e.g., traffic forecast and length of facilities) are known, incident frequency for either urban freeways or arterials can be estimated using HERS or other crash prediction methodologies. Details for each calculation and suggested default values for both evaluation types are provided later in this guide.

Unfortunately, there is no consistency across agencies' incident data recording systems. Some agencies simply record the incident duration and number of lane closures without regard to the roadway shoulder. However, the HCM freeway incident classification categorizes shoulder accident, shoulder disablement, and lane closures separately. Most of the incident databases show that shoulder closures are more frequent than lane closures. Consequently, shoulder closures should represent a significant share in the incident type distribution.

The following procedures are recommended for both data-rich and data-poor agencies to prepare and process their incident data or estimate incidents in a compatible format with the reliability analysis developed under the SHRP 2 L08 project.

Best Data Quality: Local Incident Data

This option is for data-rich agencies wishing to evaluate existing facilities. For any analysis period, incident occurrence can be estimated through a simple probability calculation. At least one full year of detailed incident data will be required. To customize the incident input tables (see Table D.8), the data must have information on incidents' duration and lane closure effect. If multiple years of incident data are available, the analyst should rely on the entire data set in the analysis.

To calculate the probabilities of incident occurrence in each month, the analyst should first separate the recorded incidents by type and by the month in which they occurred.

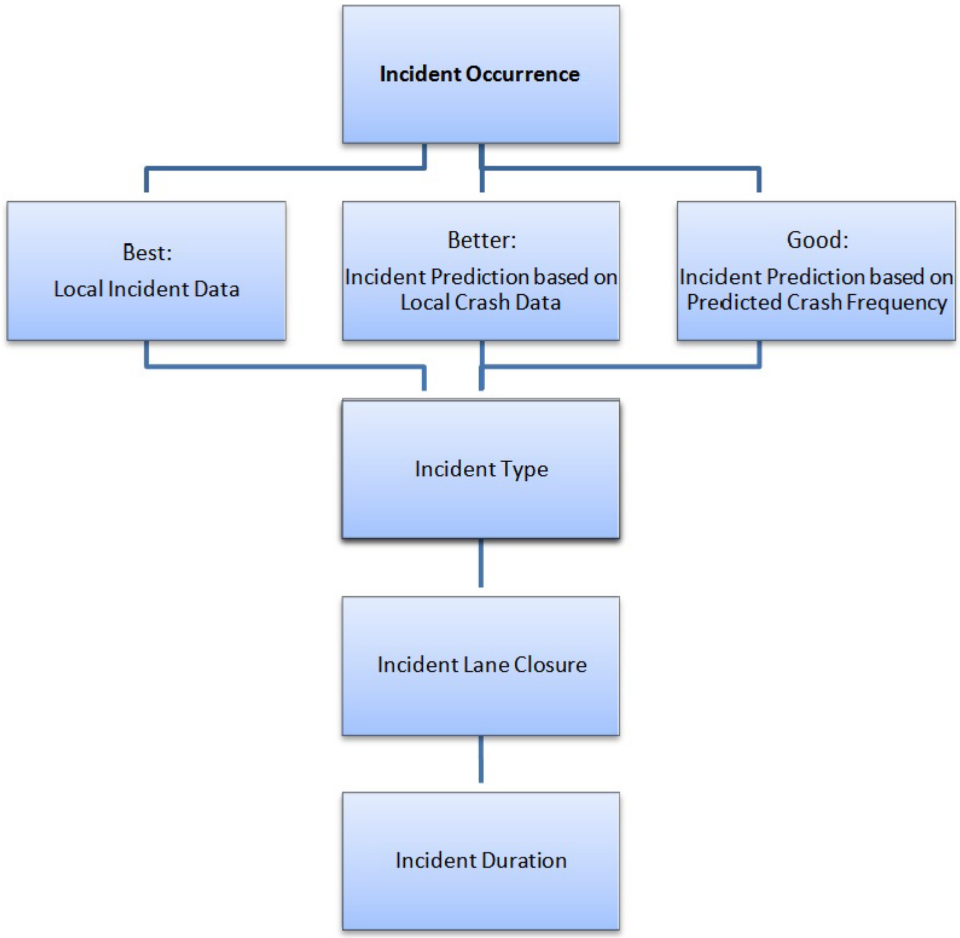


Figure D.4. Incident data processing schematic.

Table D.8. Incident Probabilities in SHRP 2 Project L08: Urban Freeways Methodology

Option B (for Data Rich Facilities): Enter Probabilities Directly in the Table from Incident Logs

By Pressing Below, the table clears to allow for direct Probability entries

Insert Facility Specific

	Probability of Different Incident Types					
Month	**No Incident**	**Shoulder Closure**	**One Lane Closure**	**Two Lane Closure**	**Three Lane Closure**	**Four Lane Closure**
January	97.59%	1.72%	0.48%	0.12%	0.09%	0.00%
February	99.58%	0.30%	0.08%	0.02%	0.02%	0.00%
March	97.55%	1.75%	0.48%	0.12%	0.10%	0.00%
April	99.23%	0.55%	0.15%	0.04%	0.03%	0.00%
May	97.32%	1.91%	0.53%	0.13%	0.10%	0.00%
June	98.17%	1.31%	0.36%	0.09%	0.07%	0.00%
July	97.59%	1.72%	0.48%	0.12%	0.09%	0.00%
August	98.60%	1.00%	0.28%	0.07%	0.05%	0.00%
September	96.94%	2.19%	0.60%	0.15%	0.12%	0.00%
October	99.60%	0.29%	0.08%	0.02%	0.02%	0.00%
November	98.73%	0.91%	0.25%	0.06%	0.05%	0.00%
December	99.62%	0.27%	0.08%	0.02%	0.01%	0.00%

A detailed explanation of categorizing incidents by the five lane closure effects is provided in this appendix. For each month of the year ($m = 1, 2, 3, \ldots, 12$), the analyst should calculate the probability of occurrence of each of the five incident types ($i = 1, 2, \ldots, 5$). The calculation is based on the proportion of the analysis period that the facility was subject to each type of incident as shown in Equation D.3:

$$\text{Probability}_{i,m} = \frac{\sum_{n=1}^{n=N_{i,m}} \text{Incident duration}_n}{\text{Length of analysis period}} \tag{D.3}$$

where $N_{i,m}$ is the number of incidents of type i in month m, and the length of analysis period is the total duration of the analysis period (e.g., a 3-h p.m. peak analysis period should have a total monthly duration of [number of weekdays in month $i \times 3$ h]).

For the probability of a shoulder closure in January (p.m. peak analysis), for example, the analyst may replace the default value with the sum of the durations of all shoulder closure incidents occurring in January during the p.m. peak, divided by the length of the p.m. peak times the number of weekdays in January.

If incident duration is not known, the analyst may use the product of the monthly incident counts and the average durations in place of the summation of incident durations. Table D.9 shows national defaults for incident data.

Better Incident Data Quality: Expansion of Crash Data into Incident Data

This option is appropriate for data-poor agencies in the evaluation of existing facilities. Data-poor agencies are those that do not routinely collect incident data or have no access to incident data. In absence of complete incident data, these agencies can use crash data for estimating incidents within the same facility. Then, they can use average durations to compute monthly incident probabilities.

Table D.9. National Defaults for Incident Data from SHRP 2 Project L08, Freeways

Incident Type	Incident Type Distribution (%)	Expected Duration (min)	SD of Duration
Shoulder closure	75.40	32	15
One-lane closure	19.60	34	14
Two-lane closure	3.10	53	14
Three-lane closure	1.90	69	22
Four-lane closure	0.00	69	22

Note: SD = standard deviation.

Table D.10. Crash-to-Incident Factors

Facility Type	Crash-to-Incident Factor
Freeways	
Range	2.4–15.4
Average	4.9
Median	6.5
Arterials	
Range	2.8–3.3
Average	3.0
Median	2.9

Total number of crashes, regardless of crash severity type, can be used for estimating a frequency of total incidents for the same facility and time period.

A crash-to-incident factor was developed based on crash proportions in various incident data sets. On average, crashes account for approximately 20.5% of all incidents or, stated another way, the number of incidents is about 4.9 times the number of crashes. These factors are shown in Table D.10.

The crash-to-incident factors can be used to populate total incident frequency for the evaluated facility type by directly multiplying them by number of crashes for the evaluated facility. For flexibility of application, a range, average, and median are provided.

When the local crash option is used to estimate total incident frequency, the analyst can further identify the probabilities of occurrence of the estimated incidents for a given time period and day.

Good Incident Data Quality: Incident Prediction Based on Predicted Crash Frequency

This option is appropriate for data-poor agencies for the evaluation of existing facilities. It can also be used by both data-poor and data-rich agencies for the evaluation of future facilities, when there is no crash history at a site.

In the absence of crash records, an expected total number of crashes can be estimated using one of the following available prediction tools:

- HSM crash prediction models (American Association of State Highway and Transportation Officials 2010);
- HERS crash prediction models (Federal Highway Administration 2005); or
- Crash rate (crashes per MVMT).

Each of the methods above requires a different set of inputs for traffic and geometry information. Depending on the facility type being evaluated, the HSM or HERS methods may be applicable. At this time, only the HERS method can be used to predict crash rates in freeways. In other words, the HSM methods provide crash prediction models for arterials only, but HERS does so for both freeways and arterials. A freeway model is currently being developed by the HSM, and should be considered when available.

Crash rates can also be considered for both freeways and arterials. The rates are available from the ITS Deployment Analysis System (IDAS) for crash prediction by crash severity type for freeways and arterials based on the known volume-to-capacity (v/c) ratio (Cambridge Systematics 2003). Alternatively, total crash frequency for freeways can be obtained from the crash rates based on the known scale for traffic speeds (Yeo et al. 2013).

Table D.11 provides a list of related crash prediction models for both facility types. More details for these prediction methods can be found in the references provided in each of the prediction tools.

Similar to the better incident data quality example, the expected number of crashes from the prediction methods can be factored up to estimate the total number of incidents by using the crash-to-incident factors provided in Table D.10. However, it is recommended that no further

Table D.11. Crash Prediction Methods

<table>
<tr><th>Facility Type</th><th>Crash Prediction Tool</th><th>Comments</th></tr>
<tr><td rowspan="2">Urban Freeways</td><td>Option 1: Arterial crash prediction using HSM models:
Go to Chapter 12 of HSM, apply Equation 12-8.</td><td>Estimate number of crashes per 100 million VMT (100MVMT)</td></tr>
<tr><td>Option 2: Freeway crash prediction by crash rates
<table>
<tr><th>Crash Severity Type</th><th>V/C or Traffic Conditions</th><th>Crash Rate (Crashes/MVMT)</th></tr>
<tr><td>Fatal</td><td>0.09–1.00</td><td>0.0066</td></tr>
<tr><td rowspan="4">Injury</td><td>0.09–0.69</td><td>0.4763</td></tr>
<tr><td>0.70–0.89</td><td>0.5318</td></tr>
<tr><td>0.90–0.99</td><td>0.6770</td></tr>
<tr><td>1.00</td><td>0.7060</td></tr>
<tr><td rowspan="4">Property damage only (PDO)</td><td>0.09–0.69</td><td>0.6171</td></tr>
<tr><td>0.70–0.89</td><td>0.7183</td></tr>
<tr><td>0.90–0.99</td><td>0.8365</td></tr>
<tr><td>1.00</td><td>0.9192</td></tr>
<tr><td rowspan="4">All severity types</td><td>FF</td><td>0.72</td></tr>
<tr><td>BN</td><td>4.43</td></tr>
<tr><td>BQ</td><td>4.48</td></tr>
<tr><td>CT</td><td>4.55</td></tr>
</table>
Source: Adapted from Tables B.2.10 through B.2.12 of IDAS User's Manual (Cambridge Systematics 2003) and Yeo et al. (2013).
Note: FF = free-flow (upstream and downstream speeds >50 mph); BN = bottleneck (downstream speed >50 mph, upstream speed <50 mph); BQ = back-of-queue (downstream speed <50 mph, upstream speed >50 mph); and CT = congested (upstream and downstream speeds <50 mph).</td><td>Estimate number of crashes per million VMT (MVMT)</td></tr>
<tr><td rowspan="3">Urban Arterials</td><td>Option 1: Arterial Crash Prediction by HSM Models (Chapter 12)

Use HSM Equation 12-8</td><td>Estimate number of crashes for the period of interest</td></tr>
<tr><td>Roadway segment crashes are a combination of the following predictions:</td><td></td></tr>
<tr><td>Multiple-vehicle nondriveway crashes
Use HSM Equation 12-10</td><td></td></tr>
</table>

(continued on next page)

Table D.11. Crash Prediction Methods (continued)

<table>
<tr><th>Facility Type</th><th>Crash Prediction Tool</th><th>Comments</th></tr>
<tr><td>Urban Arterials</td><td>Single-vehicle crashes
Use HSM Equation 12-13</td><td></td></tr>
<tr><td></td><td>Multiple-vehicle driveway-related crashes
Use HSM Equation 12-16</td><td></td></tr>
<tr><td></td><td>Vehicle–pedestrian crashes
Use HSM Equation 12-19</td><td></td></tr>
<tr><td></td><td>Vehicle–bicycle crashes
Use HSM Equation 12-20</td><td></td></tr>
<tr><td></td><td>Intersection crashes are combination of the following predictions:</td><td></td></tr>
<tr><td></td><td>Vehicle–vehicle crashes for intersections
Use HSM Equation 12-21
Use HSM Equation 12-20</td><td></td></tr>
<tr><td></td><td>Vehicle–pedestrian crashes for signalized intersections
Use HSM Equation 12-28
Use HSM Equation 12-29</td><td></td></tr>
<tr><td></td><td>Vehicle–pedestrian crashes for stop-controlled intersections
Use HSM Equation 12-30</td><td></td></tr>
<tr><td></td><td>Vehicle-Bicycle Crashes
Use HSM Equation 12-31</td><td></td></tr>
<tr><td></td><td>Option 2: Arterial Crash Prediction by HERS Models (Chapter 5)

Urban Multilane Highway Crashes

HERS Equation 5.34: Crash rate = $A \times AADT^{B} \times NSIGPM^{C}$

where
Crash rate = number of crashes per 100 million vehicle miles on urban multilane highways;
NSIGPM = number of signals per mile ($0.1 \le NSIGPM \le 8$); and
A, B, and C = coefficients provided below.

<table>
<tr><th>Type of Section</th><th>A</th><th>B</th><th>C</th></tr>
<tr><td>Two-way with left-turn lane</td><td>95.0</td><td>0.1498</td><td>0.4011</td></tr>
<tr><td>One-way, or two-way with a median</td><td>82.6</td><td>0.1749</td><td>0.2515</td></tr>
<tr><td>(1) wider than 4 ft,</td><td></td><td></td><td></td></tr>
<tr><td>(2) cubed, or</td><td></td><td></td><td></td></tr>
<tr><td>(3) a “positive barrier”</td><td></td><td></td><td></td></tr>
<tr><td>Otherwise</td><td>115.8</td><td>0.1749</td><td>0.2515</td></tr>
</table>
Source: HERS Table 5-9 (FHWA 2005).</td><td>Estimate number of crashes per 100 million VMT (100MVMT) on urban multilane highways</td></tr>
<tr><td></td><td>Urban Two-Lane Highway Crashes

HERS Equation 5.36: Crash Rate = $-19.6 \times \ln(AADT) + 7.93 \times (\ln[AADT])^2$

where crash rate = number of crashes per 100 million vehicle miles on two-lane streets.</td><td>Estimate number of crashes per 100 million VMT (100MVMT) on urban two-lane highways</td></tr>
<tr><td></td><td>Option 3: Arterial Crash Prediction by Crash Rates

<table>
<tr><th>Crash Severity Type</th><th>V/C or Traffic Conditions</th><th>Crash Rate (Crashes/MVMT)</th></tr>
<tr><td>Fatal</td><td>0.09–1.00</td><td>0.0177</td></tr>
<tr><td>Injury</td><td>0.09–1.00</td><td>1.6991</td></tr>
<tr><td>Property damage only</td><td>0.09–1.00</td><td>2.4736</td></tr>
</table>
Source: Adapted from Tables B.2.10 through B.2.12 of IDAS User’s Manual (Cambridge Systematics 2003).</td><td>Estimate number of crashes per million VMT (MVMT)</td></tr>
</table>

treatment—including custom occurrence probabilities, free-flow speed adjustment, or capacity adjustments—be made on HERS estimates.

Estimation of Incident Probabilities

When the total incident frequency is estimated using the methodologies in either the better or good data-quality option, the analyst can further identify probabilities of incident occurrence by using the suggested probabilities provided in Table D.12 and Table D.13. Table D.12 provides the probabilities of having or not having incidents for each analysis month. Table D.13 provides the probabilities of incident occurrence within each peak period for each month.

The probabilities provided in Table D.12 can be used for different purposes. They can be directly applied to estimate incident frequency for each peak period in a given month. They can also be used in conjunction with Table D.13 to populate the probability of incident occurrence for a given peak period and a given month.

The level of analysis for incident occurrence should be peak periods. The incident probabilities in Table D.13 are provided for the three time periods: 6 to 11 a.m., 3 to 8 p.m., and off-peak periods. Thus, the probabilities provided are for the estimate of total incidents occurring within 5 h for the a.m. peak period, 5 h for the p.m. peak period, and 14 h for the off-peak period.

The following example illustrates the use of Table D.13 to estimate incident frequency for a given peak period of each month. When incident frequency is identified in number of incidents per month, a number of incidents broken down by peak period can be obtained using the probabilities in Table D.13. For example, if a freeway facility is projected to have 20 incidents per month, the probability of incident occurrence from 3 to 8 p.m. in January is approximately

Table D.12. Incident Occurrence Probabilities for Each Month

Month	Freeway	
	Having Incidents	**No Incidents**
January	0.9032	0.0968
February	0.8839	0.1161
March	0.9194	0.0806
April	0.9000	0.1000
May	0.8871	0.1129
June	0.8750	0.1250
July	0.9113	0.0887
August	0.8871	0.1129
September	0.9000	0.1000
October	0.7903	0.2097
November	0.8417	0.1583
December	0.8790	0.1210
Total	0.8815	0.1185

Table D.13. Incident Occurrence Probabilities by Peak Period

Month	Freeway				Arterial			
	6–11 a.m.	**3–8 p.m.**	**Off Peak**	**Total**	**6–11 a.m.**	**3–8 p.m.**	**Off Peak**	**Total**
January	0.2991	0.3870	0.3139	1.0000	0.2737	0.4031	0.3235	1.0000
February	0.2509	0.4514	0.2977	1.0000	0.2754	0.4233	0.3009	1.0000
March	0.2643	0.4400	0.2958	1.0000	0.2742	0.4402	0.2857	1.0000
April	0.2593	0.4124	0.3283	1.0000	0.2770	0.3803	0.3427	1.0000
May	0.2411	0.4456	0.3132	1.0000	0.2788	0.3772	0.3444	1.0000
June	0.2548	0.4185	0.3268	1.0000	0.3268	0.3756	0.2975	1.0000
July	0.2569	0.4455	0.2978	1.0000	0.3146	0.4272	0.2583	1.0000
August	0.2402	0.4203	0.3396	1.0000	0.2592	0.4260	0.3149	1.0000
September	0.2581	0.4227	0.3192	1.0000	0.2312	0.3869	0.3820	1.0000
October	0.2386	0.4522	0.3093	1.0000	0.3331	0.3784	0.2881	1.0000
November	0.2382	0.4538	0.3079	1.0000	0.3037	0.3878	0.3083	1.0000
December	0.2256	0.4441	0.3302	1.0000	0.4429	0.0498	0.5072	1.0000

Table D.14. Average, Median, and Standard Deviation of Incident Type Probabilities

Facility Type	Incident Distribution (%)			
	Crash	Breakdown	Debris	Other
Freeway				
Range	6.5–41.4	45–88.6	1.1–13.2	0.4–7.5
Average	20.5	69.7	5.9	3.9
Median	15.4	74.6	5.8	4.2
SD	13.1	15.6	4.3	3.1
Sample Size	18,206	76,758	7,813	737
Arterial				
Range	30.2–35.6	27.3–58.8	5.2–7.8	4–37.8
Average	32.9	45.3	6.4	15.5
Median	35.0	52.7	6.5	5.8
SD	2.9	16.7	1.8	19.0
Sample Size	1,733	1,757	205	958

20 (incidents) × 0.3870 (Table D.13, Freeways, January, 3–8 p.m.) = 7.74, or eight incidents.

To identify the probability of incident occurrence for each peak period and for each month, the analyst can calculate a product of the probabilities from Table D.12 and Table D.13. For example, the probability of having incidents on a freeway facility from 3 to 8 p.m. in January is 0.9032 (Table D.12, Freeways, January) × 0.3870 (Table D.13, Freeways, January, 3–8 p.m.) = 0.3495. These can further be refined by incident type with the help of Table D.14.

Determine the Type of Lane Closure Effect

The lateral lane closure is considered in HCM2010 as one of the parameters that has a direct effect on freeway capacity. Closing more lanes due to incidents significantly decreases the service capacity of freeways.

The majority of recorded incidents close the shoulder lane more frequently than travel lanes. The likelihood of the lane closure due to incidents declines with the number of lanes. For example, incident data analysis shows that approximately 4% of all incidents close two lanes. The proportion of lane closure for three and more lanes is approximately 2%. If lane closure information is further stratified by incident types, the lane closure proportion for three and more lanes would be even lower.

Five lateral lane closure effects are recommended in the SHRP 2 Project L08 freeways reliability analysis. These include

- Shoulder closure;
- 1-Lane closure;
- 2-Lane closure;
- 3-Lane closure; and
- 4-Lane closure.

Urban streets methodology only considers three lane-closure types: shoulder, one lane, and two or more lanes.

Data-Rich Agencies

Similar to the incident occurrence analysis, the analyst can determine incident lane closure types in two ways. The first option is by turning incident-log data into a cumulative distribution function by lane closure for each incident type and randomly sampling from it. Alternatively, it can be done by turning incident logs into a frequency distribution by lane closure type for each incident type and identifying a percentage distribution across all lane closure types.

Data-Poor Agencies

In the absence of local incident-log data, data-poor agencies can use the default probabilities of incident occurrence by lane closure type provided in both the urban freeways and urban streets tool. Common lane closure types found in various incident data sets include shoulder, one lane, two lanes, and three or more lanes. These are consistent with the lane closure types used in the HCM. Some agencies, such as the Washington State Department of Transportation, record multiple lanes without separately specifying two-lane or three-lane.

Table D.15 provides a greater detail of the default incident distribution by lane closure types in the urban freeways and urban streets tools. The analyst should be aware that the urban freeways methodology will require only incident distribution by lane closure type, while the urban streets methodology will require incident distribution by incident type or crash severity.

In the urban streets tool, the incident is broken down by crash- and noncrash-related type. The analyst can use the default values provided in the urban streets tool, which is presented in greater detail in Table D.16, to identify the lane closure type for property-damage-only and injury or fatal crash types.

Table D.15. Lane Closure Type Distribution by Incident Type

Facility Type	Incident Type	Statistic	Lane Closure Distribution (%)			
			Shoulder	1 Lane	2 Lanes	3 or More Lanes
Freeway	Crash	Range	12.3–79	15.6–44.4	0.7–25.1	2–18.2
		Average	55.8	27.8	9.4	7.0
		Median	59.1	27.0	8.3	5.5
		SD	19.8	8.1	7.5	5.2
		Sample size	6,825	5,749	2,512	1,736
	Breakdown/disabled/stalled	Range	3–98.3	1.6–92.8	0.1–1.3	0–3.5
		Average	81.0	17.9	0.5	0.7
		Median	91.7	7.9	0.2	0.2
		SD	30.2	28.9	0.4	1.3
		Sample size	63,292	8,308	353	214
	Debris	Range	0.9–96.2	3.3–88	0.3–21.4	0–8.8
		Average	40.3	51.2	6.1	2.5
		Median	28.5	66.6	3.1	1.7
		SD	36.5	32.1	7.1	3.0
		Sample size	1,326	3,011	439	156
	Other	Range	43.2–95.9	3.5–45.8	0.4–4.2	0.2–9.7
		Average	66.0	27.5	2.2	4.2
		Median	63.4	30.8	2.2	3.5
		SD	22.2	18.3	1.7	4.2
		Sample size	376	150	15	28
Arterial	Crash	Range	37.4–50.5	34.4–51.3	0–9.6	1.7–15.1
		Average	44.0	42.9	4.8	8.4
		Median	44.0	42.9	4.8	8.4
		SD	9.3	12.0	6.8	9.5
		Sample size	360	275	11	97
	Breakdown/disabled/stalled	Range	2.8–77.2	22.8–85.2	0–0.3	0–11.6
		Average	40.0	54.0	0.2	5.8
		Median	40.0	54.0	0.2	5.8
		SD	52.6	44.1	0.2	8.2
		Sample size	199	842	3	108
	Debris	Range	0–45	50–89.1	2.2–5	0–8.8
		Average	22.5	69.5	3.6	4.4
		Median	22.5	69.5	3.6	4.4
		SD	31.8	27.6	2.0	6.2
		Sample size	9	132	5	12
	Other	Range	13.6–15.5	50–59.2	0–27.3	9.1–25.4
		Average	14.5	54.6	13.6	17.2
		Median	14.5	54.6	13.6	17.2
		SD	1.3	6.5	19.3	11.5
		Sample size	14	67	10	21

Table D.16. Lane Closure Type by Crash Severity

Facility Type	Crash Severity Distribution (%)							
	Shoulder		1 Lane		2 Lanes		3 or More Lanes	
	PDO	I + F	PDO	I + F	PDO	I + F	PDO	I + F
Freeway								
Range	85.7–96.6	3.4–14.3	41.7–100	0–58.3	0–100	0–100	58.9–100	0–41.1
Average	90.0	10.0	74.4	25.6	58.9	41.1	77.4	22.6
Median	89.5	10.5	75.7	24.3	72.2	27.8	75.3	24.7
SD	0.04	0.04	0.23	0.23	0.38	0.38	0.18	0.18
Sample size	2,563	247	3,104	482	1,158	382	710	448
Arterial								
Range	88.0–97.4	2.6–12.0	67.1–92.9	7.1–32.9	58.3–72.7	27.3–41.7	100.0	0.0
Average	92.7	7.3	80.0	20.0	65.5	34.5	100.0	0.0
Median	92.7	7.3	80.0	20.0	65.5	34.5	100.0	0.0
SD	0.07	0.07	0.18	0.18	0.10	0.10	na	na
Sample size	317	39	197	75	50	33	1	0

Note: PDO = property damage only; I + F = injury and fatal.

Determine Incident Duration

An unplanned incident typically begins at the time it is entered into the system and formally ends when all involved vehicles are off the shoulder or when the last related activity is recorded in the system. Some agencies may consider incident duration from different clearance stages, such as when the obstruction is removed, when the lanes are reopened, or when traffic returns to its normal stage.

Data-Rich Agencies

Similar to the previous analyses, the analyst can determine incident durations by using two options. The first option is by turning incident-log data into a distribution function by incident duration for each incident type and lane closure type and randomly sampling from it. Alternatively, it can be done by identifying average incident durations by incident type and by lane closure type from the incident-log data.

If incident duration is sampled from distributions, it is recommended that the distribution be truncated on both ends to avoid very small (e.g., 1-min) or very large incident durations. The advantage of using a formal probability distribution (rather than creating a customized cumulative distribution function from the field data) is that the effect of incident management strategies can be tied to the mean duration, thus not requiring that the entire cumulative distribution function be retuned.

Depending on the methodology used for either urban freeway or street, the same incident types and lane closure types as suggested in the previous analyses should be considered in identifying incident durations. The lane closure type used in the urban freeways methodology include shoulder closure, one-lane closure, two-lane closure, three-lane closure, and four-lane closure. The urban streets methodology considers three types of lane closure, including shoulder closure, one-lane closure, and two-lane closure.

Data-Poor Agencies

In the absence of incident-log data, it is recommended that the incident durations by incident type and lane closure be estimated using the default values provided in the urban freeways and urban streets tools.

Table D.17 provides greater detail of the default incident durations in the urban freeways and urban streets tools. The analyst should be aware that the urban freeways methodology will require only incident duration by lane closure type, while the urban streets methodology will require incident duration by lane closure type and incident type.

In the urban streets tool, the incident duration is broken down by severity type. The analyst can use the default values provided in the urban streets tool, which is presented in greater detail in Table D.18, to identify the duration for property-damage-only and injury or fatal crash types.

Table D.17. Incident Duration by Incident Type and Lane Closure Type

Facility Type	Incident Type	Statistic	Incident Duration (min)			
			Shoulder	1 Lane	2 Lanes	3 or More Lanes
Freeway	Crash	Range	20.5–69.5	32.9–59.3	31–73.7	30.7–97.4
		Average	43.1	45.1	58.9	71.9
		Median	40.4	38.9	66.2	75.8
		SD	14.7	10.5	15.0	22.7
		Sample size	6,099	5,476	2,424	1,685
	Breakdown/disabled/stalled	Range	7.2–54.1	16–58.1	23.5–72	16.8–263.1
		Average	29.7	30.1	46.1	73.5
		Median	27.4	27.8	42.1	40.0
		SD	15.1	13.7	14.7	94.1
		Sample size	59,606	8,043	339	211
	Debris	Range	8.1–76	6.7–53	14.1–83.6	15.4–54
		Average	35.7	25.5	40.8	32.1
		Median	35.7	21.5	32.1	26.5
		SD	25.0	14.6	25.7	16.8
		Sample size	1,242	2,925	431	156
	Other	Range	4.5–38.6	16.5–76.9	12–146.2	2.3–61.8
		Average	26.0	46.8	77.3	40.8
		Median	30.4	46.9	73.6	58.3
		SD	15.1	24.7	67.2	33.4
		Sample size	290	119	8	17
Arterial	Crash	Range	18.8–39.9	30.6–47	44.3–44.3	43.2–45.2
		Average	29.3	38.8	44.3	44.2
		Median	29.3	38.8	44.3	44.2
		SD	14.9	11.6	N/A	1.4
		Sample size	360	275	11	97
	Breakdown/disabled/stalled	Range	18.8–32	10.2–24.4	17–17	26.4–26.4
		Average	25.4	17.3	17.0	26.4
		Median	25.4	17.3	17.0	26.4
		SD	9.3	10.1	N/A	N/A
		Sample size	199	842	3	108
	Debris	Range	57.7–57.7	16.2–25.3	1.1–22	32.6–32.6
		Average	57.7	20.8	11.6	32.6
		Median	57.7	20.8	11.6	32.6
		SD	N/A	6.5	14.8	N/A
		Sample size	9	132	5	12
	Other	Range	41.9–47	51.4–70.8	123.9–123.9	9.2–65.8
		Average	44.4	61.1	123.9	37.5
		Median	44.4	61.1	123.9	37.5
		SD	3.6	13.7	0.0	40.0
		Sample size	14	67	10	21

Note: N/A = not applicable.

Table D.18. Incident Duration by Crash Severity Type

Facility Type	Duration by Crash Severity Type (min)							
	Shoulder		1 Lane		2 Lanes		3 or More Lanes	
	PDO	I + F	PDO	I + F	PDO	I + F	PDO	I + F
Freeway								
Range	18.1–65	42–94	27.4–61	40.9–52	31–78	32–72	29–76	29–76.9
Average	38.1	60.7	42.3	46.4	56.4	54.7	57.8	45.0
Median	36.0	48.0	37.8	48.0	58.4	53.5	63.1	29.0
SD	17.1	22.2	14.0	4.4	22.7	16.8	20.4	27.7
Sample size	2,563	247	3,104	482	1,158	382	710	448
Arterial								
Range	17–38	30–57	30–46	42–50	31–46	39–55	22–22	N/A
Average	27.5	43.5	38.0	46.0	38.5	47.0	22.0	N/A
Median	27.5	43.5	38.0	46.0	38.5	47.0	22.0	N/A
SD	14.8	19.1	11.3	5.7	10.6	11.3	N/A	N/A
Sample size	317	39	197	75	50	33	1	0

Note: N/A = not applicable.

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APPENDIX E

Weather-Modeling Alternatives and Validation for the Freeway and Urban Street Scenario Generators

Weather is one of the seven sources of congestion on the transportation network. Weather has a twofold effect on roadways: influencing driver behavior and increasing the likelihood of incidents (another source of congestion). Recent studies have attempted to separate the sources of unreliable travel with varying contributions compared with other sources of congestion, but all point to weather as significantly contributing to unreliable travel. Figure E.1 shows a typical breakdown of the sources of unreliable traffic, which shows weather effects at 15%.

This appendix describes typical sources of weather data used in transportation studies and the attributes they monitor. Two methods for predicting weather events or probabilities are described, and each is validated against historical weather data. Both methodologies are discussed with respect to their applicability for predicting weather impacts on travel time reliability in the context of SHRP 2 Project L08.

Weather Data Sources

Various national sources for weather data can be used to obtain historical data and trends for weather patterns for different locations across the United States. Some of the main sources are discussed below in light of their applicability to this project.

CLARUS Initiative

The CLARUS initiative was established in 2004 by the Federal Highway Administration's Road Weather Management Program in conjunction with the Intelligent Transportation System Joint Program Office. The main goal of the CLARUS initiative was to "create a robust data assimilation, quality checking, and data dissemination system that could provide near real-time atmospheric and pavement observations from collective state's investments in road weather information system, environmental sensor stations as well as mobile observations from Automated Vehicle Location equipped trucks."

Weather data can be obtained from the CLARUS system by (1) subscribing to a report that is periodically updated, (2) using the latest quality-checked observations on the map interface, and (3) retrieving an on-demand report for weather observations (Federal Highway Administration 2012).

National Climatic Data Center

The National Climatic Data Center of the National Oceanic and Atmospheric Administration periodically publishes climatic data summaries from weather stations in each of 284 U.S. cities and territories (National Climatic Data Center 2011). The published document contains 17 statistics related to temperature, wind, cloudiness, humidity, and precipitation. Each statistic is quantified by month of year and based on 10 or more years of data.

The National Climatic Data Center also provides storm event data for several thousand locations throughout the United States (including the 284 cities previously mentioned). These data describe the average number of storms, average precipitation depth per storm, average storm duration, and average rainfall rate (i.e., intensity). Each statistic is quantified by month of year.

National Weather Service

The National Weather Service (NWS) of the National Oceanic and Atmospheric Administration is tasked with monitoring weather across the United States. The NWS provides real-time weather data as well as short-term weather predictions. Real-time reports are primarily from meteorological aviation reports (METARs) from airport weather stations across the United States that contain weather information vital to pilots including wind, visibility, weather type, cloud, temperature, and pressure data. They are typically reported hourly or every 30 min at 50 min past each hour. Special reports (SPECI) are reported when there is a significant change in the weather

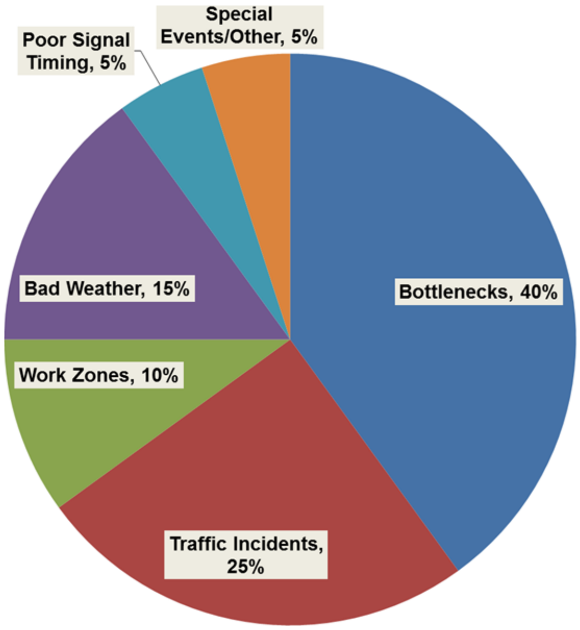

Figure E.1. Sources of congestion.

that occurs between scheduled hourly transmissions. A SPECI will be issued if any of the following occurs:

- The ceiling decreases to 1,500 ft or less, or when a cloud layer, previously not reported, appears below 1,000 ft (or below the highest minimum for straight-in instrument flight rules [IFR] landings, or the minimum for IFR departures);
- Visibility decreases to below 3 statute miles;
- A tornado, waterspout, or funnel cloud is reported;
- A thunderstorm begins, intensifies to "heavy," or ends;
- Precipitation begins, changes, or ends;
- Winds suddenly increase and exceed 30 knots (speed must double), or when the direction of the winds significantly changes (satisfying the criteria for "wind shift").

Weather Underground (www.wunderground.com) is a weather data service that archives METARs and SPECI for all U.S. NWS weather stations. The service is provided for free and is used by many other services that require historical weather data, such as Wolfram Mathematica.

Attributes of Weather Data

The following lists indicate the weather attributes that are reported by each of the data sources described above.

CLARUS

- Air temperature;
- Dew point;
- Relative humidity;
- Surface status;
- Surface temperature;
- Precipitation intensity;
- Precipitation type;
- Wind direction;
- Wind speed;
- Wind gust direction; and
- Wind gust speed.

National Climatic Data Center

Among different weather statistics the following items are of interest to reliability evaluation:

- Mean number of days with precipitation of 0.01 in. or more;
- Total snowfall;
- Normal daily mean temperature; and
- Normal precipitation.

National Weather Service

- Wind direction;
- Wind speed;
- Visibility;
- Weather type or phenomena;
- Cloud amount;
- Cloud height;
- Temperature;
- Dew point;
- Pressure; and
- Precipitation.

Weather Effects on Traffic

Weather has a significant impact on the operations of both freeway and urban street facilities. Past research has been performed to quantify the effects of weather by category. An extensive review of the literature on weather impacts on traffic operations has been completed by the SHRP 2 L08 project team that has been summarized in a separate white paper. This section presents a summary of the key findings.

Weather affects operations on both freeway and urban street facilities, although the literature is more extensive on weather impacts on freeways. These facility types are discussed separately in the following subsections.

Weather Impacts on Freeways

Most research on weather effects on freeways has focused on capacity. The *2010 Highway Capacity Manual* (HCM2010) (Transportation Research Board of the National Academies

Table E.1. Summary of Weather Capacity and Speed Adjustment Factors for Freeways

Weather Type		Capacity Adjustment Factors (CAF)					Free-Flow Speed Adjustment Factors (SAF)				
Free-Flow Speed (mph)*		55 mph	60 mph	65 mph	70 mph	75 mph	55 mph	60 mph	65 mph	70 mph	75 mph
Clear	Dry Pavement	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	Wet Pavement	0.99	0.98	0.98	0.97	0.97	0.97	0.96	0.96	0.95	0.94
Rain	≤0.10 in/h	0.99	0.98	0.98	0.97	0.97	0.97	0.96	0.96	0.95	0.94
	≤0.25 in/h	0.94	0.93	0.92	0.91	0.90	0.96	0.95	0.94	0.93	0.93
	>0.25 in/h	0.89	0.88	0.86	0.84	0.82	0.94	0.93	0.93	0.92	0.91
Snow	≤0.05 in/h	0.97	0.96	0.96	0.95	0.94	0.94	0.92	0.89	0.87	0.84
	≤0.10 in/h	0.95	0.94	0.92	0.90	0.88	0.92	0.90	0.88	0.86	0.83
	≤0.50 in/h	0.93	0.91	0.90	0.88	0.87	0.90	0.88	0.86	0.84	0.82
	>0.50 in/h	0.80	0.78	0.76	0.74	0.72	0.88	0.86	0.85	0.83	0.81
Temp	<50 deg F	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.98	0.98
	<34 deg F	0.99	0.99	0.99	0.98	0.98	0.99	0.98	0.98	0.98	0.97
	<-4 deg F	0.93	0.92	0.92	0.91	0.90	0.95	0.95	0.94	0.93	0.92
Wind	<10 mph	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	≤20 mph	0.99	0.99	0.99	0.99	0.99	0.99	0.98	0.98	0.97	0.96
	>20 mph	0.99	0.99	0.99	0.98	0.98	0.98	0.98	0.97	0.97	0.96
Visibility	<1 mi	0.90	0.90	0.90	0.90	0.90	0.96	0.95	0.94	0.94	0.93
	≤0.50 mi	0.88	0.88	0.88	0.88	0.88	0.95	0.94	0.93	0.92	0.91
	≤0.25 mi	0.90	0.90	0.90	0.90	0.90	0.95	0.94	0.93	0.92	0.91

2010) includes 15 weather categories with an average and range of capacity effects on freeways. More recent research has also included free-flow effects. In a separate L08 project white paper a synthesis of the literature on capacity and free-flow speed (FFS) effects of weather on freeway facilities has been presented. Table E.1 summarizes the effects for freeways.

The factors in Table E.1 can be used to estimate a modified speed–flow relationship for a basic freeway segment. HCM2010 Equation 25-1 uses the capacity adjustment factor (CAF) to fit a speed–flow curve between the FFS and the newly estimated capacity. With the introduction of the speed adjustment factor (SAF) in this project, the revised equation, Table E.2, was adapted from HCM2010.

The equation can be used to estimate the speed–flow relationships for different weather events. Figure E.2 shows some illustrative examples. The figure shows the speed–flow relationships for FFS of 75, 65, and 55 mph. The impacts of medium rain and heavy snow are shown for each base FFS in dashed and dotted lines, respectively. The 45 passenger cars per hour per lane (pcphpl) density line is shown to represent the level of service E to F boundary.

Figure E.2 illustrates that the SAF results in a downward shift of the speed–flow curve, while the CAF shifts the intercept with the 45 pcphpl density line. The resulting curves in between these points are intuitive, and internally consistent, provided the SAF does not drop the FFS below the speed at capacity (after applying CAF). In that case, the methodology assumes a horizontal speed–flow relationship at a fixed speed equal to speed at capacity (after CAF), thus overriding the SAF input.

For nonbasic segments (weaving and merge–diverge sections), the methodology multiplies the FFS by SAF and the segment capacity by CAF in each occurrence in the method. Details on this implementation are provided in a separate working paper.

Table E.2. Estimating Basic Segment Speed from CAF and SAF (adapted from 2010 HCM Equation 25-1)

$$S = (\mathrm{FFS} * \mathrm{SAF}) + \left[1 - e^{\ln\left((\mathrm{FFS}*\mathrm{SAF})+1-\frac{C*\mathrm{CAF}}{45}\right)*\frac{v_p}{C*\mathrm{CAF}}}\right]$$

Weather Impacts on Urban Streets

The impacts of weather on urban streets are less well defined in the HCM2010, although extensive work has been done in this project to document the state of the practice in the literature. Weather impacts on urban streets primarily affect the saturation flow rate at signalized intersections and the midsegment FFS along an extended urban street facility.

Predicting Weather Probability by Using Historical Averages

Weather events can be predicted using Monte Carlo techniques. That technique and weather-modeling procedure are described in the section on urban street scenario development in Chapter 5 of the main report. As an alternative approach to the Monte Carlo technique, historical weather averages can be used to estimate the probability of occurrence of weather

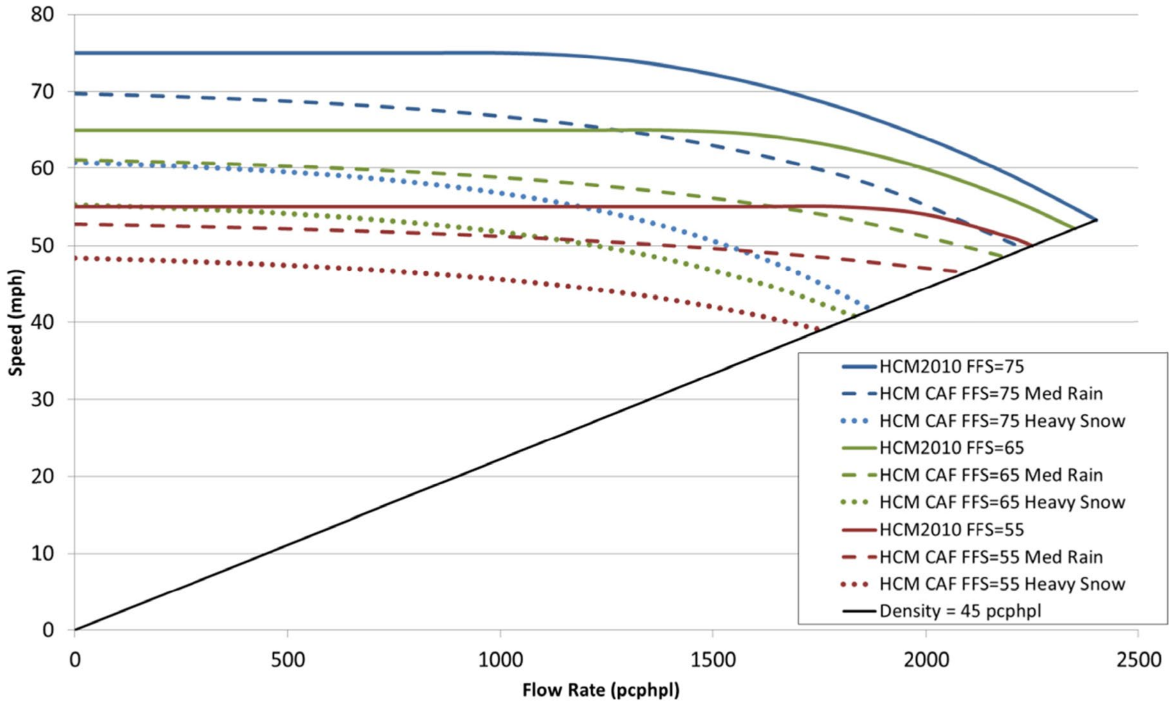

Figure E.2. Illustrative examples of CAF and SAF application for freeways.

of a certain category. This approach may seem more data intensive initially, but it eliminates any stochastic element in the application of the method. Each weather category is considered in the scenario generation process, weighted by the probability of occurrence in a particular analysis month and hour based on historical data.

Data Source and Processing

Historical average probabilities were created from NWS METARs data available from Weather Underground (2012). Historical weather is available in comma separated value (CSV) format for any airport in the Automated Surface Observing System on a daily basis. These CSV files contain all METARs for the airport and day requested. A Python (Version 2.7) script was written to automate the download of all daily CSV weather files for selected airports and years, and a second script compiles all daily files into a single CSV file containing observations from all selected years for a given airport.

Once a single CSV file for each airport was created, each file was filtered and analyzed in Microsoft Excel. There were a few issues identified with the hourly reports that had to be fixed before calculating probabilities. First, reports occasionally report "unknown" conditions, for which any field with a number is reported as "–9999." These values were changed so that they would not be picked up as a weather category with a capacity effect. Additionally, the compiler inserted reports from previous time periods infrequently. This would result in a negative duration between the previous report and the next one. Any repeated reports were removed before analysis.

For application in SHRP 2 Project L08, all weather categories outlined in the HCM2010 that reduce capacity by at least 5% are included in the probability calculation. Table E.3, taken from the HCM2010, is shown with weather categories and their associated reduction in capacity; all categories that were modeled are marked with a star. Other weather effects are considered to have minimal capacity effects and are combined with the no weather event probability.

To calculate the probability for each weather category, the duration of time between each report is calculated to account for missing reports or SPECI. This time is calculated as the time in hours between the current and previous report. Each report is then classified as one of the 10 weather categories with a capacity effect or a negligible capacity effect. If a report can be classified as having multiple categories, it is assigned to the category with the highest capacity effect. Probabilities for each category are calculated using an Excel pivot table. Each weather category is set as a column header, and the month of year and hour are set as row headers. The sum of durations in each cell is divided by the sum of the durations in each row to calculate the proportion of time for each combination of month, hour, and weather category.

Average event duration for each weather category is calculated over the 10-year data set by taking the average of all continuous

Table E.3. HCM2010 Weather Categories for Freeway Facilities

Type of Condition	Intensity of Condition	Percent Reduction in Capacity Average		Percent Reduction in Capacity Range
Rain	>0 ≤0.10 in./h	2.01		1.17–3.43
	>0.10 ≤0.25 in./h	7.24	★	5.67–10.10
	>0.25 in./h	14.13	★	10.72–17.67
Snow	>0 ≤0.05 in./h	4.29		3.44–5.51
	>0.05 ≤0.10 in./h	8.66	★	5.48–11.53
	>0.10 ≤0.50 in./h	11.04	★	7.45–13.35
	>0.50 in./h	22.43	★	19.53–27.82
Temperature	<10° C ≥1° C	1.07		1.06–1.08
	<1° C ≥-20° C	1.50		1.48–1.52
	<-20° C	8.45	★	6.62–10.27
Wind	>16 ≤32 km/h	1.07		0.73–1.41
	>32 km/h	1.47		0.74–2.19
Visibility	<1 ≥0.50 mi	9.67	★	One site
	<0.50 ≤0.25 mi	11.67	★	One site
	<0.25 mi	10.49	★	One site

Note: Stars indicate weather conditions with at least a 5% average reduction in capacity.
Source: HCM2010.

event durations. This duration is used in order to model each weather event in the computational engine.

Data Characteristics

Weather with a significant capacity effect is relatively rare in the United States, even in northern cities with significant winter weather. Figure E.3 shows the proportion of time with negligible capacity effects due to weather for airports in 40 of the largest metro areas in the United States. Las Vegas, Nevada, has the lowest probability (0.18%) of weather with a capacity effect, while Cleveland, Ohio, has the highest probability (9.33%) of the 40 largest metro areas.

Figure E.4 shows a summary of the average probability for the 10 weather categories with significant capacity effects for four major metropolitan areas: New York (KLGA); Miami, Florida (KMIA); Chicago, Illinois (KORD); and San Francisco, California (KSFO). If light rain were included in the analysis, it would greatly outweigh other factors, as would the higher temperature categories, which are very common. As shown in Figure E.5, low-intensity snow has a relatively high probability in northern cities; however, it has the lowest effect on capacity of the 10 categories included in the analysis.

Figure E.5 shows the average event duration for each of the weather categories for the same metropolitan areas. In general, the durations follow the same trend as the probabilities with one major exception. In Chicago, there was only one severe cold (<4°F) event, which lasted nearly 8 h.

Weather-Modeling Procedure

The freeway scenario generation approach in Project L08 uses the weather probabilities on a monthly basis for each category. The scenario generator contains a database of weather

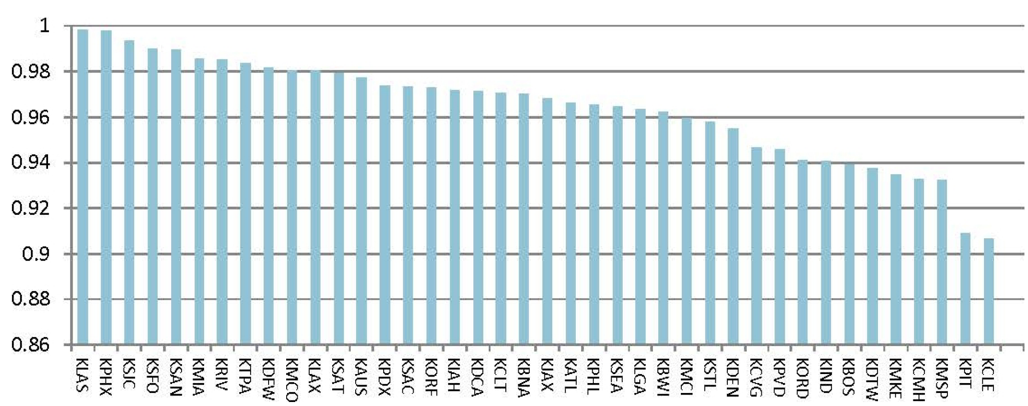

Figure E.3. Probability of weather with negligible capacity effect in 40 largest U.S. metropolitan areas, 2001–2010.

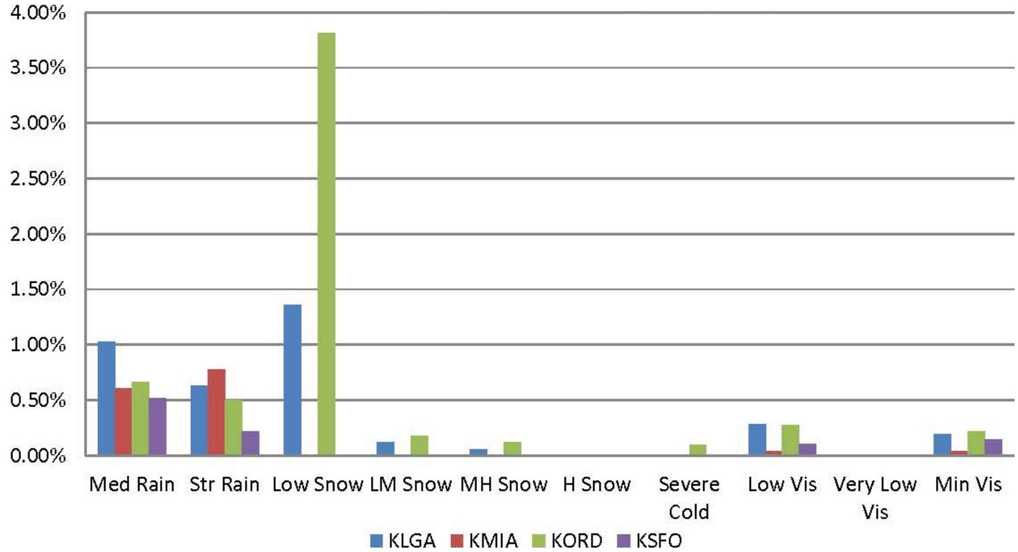

Figure E.4. Annual probability of weather types in New York, Miami, Chicago, and San Francisco airports, 2001–2010.

probabilities by month of the year and hour of day for each weather category created from the 10 years of METARs, as described earlier. Once a study period is selected, the scenario generator averages the probabilities across the study period hours (weighted by the fraction of each hour included in the study period in the case of partial hours) to create monthly probabilities by weather category. If months are grouped together (i.e., into seasons), probabilities are averaged across the months to create probabilities for each group of months by weather category (weighted by number of days in each month included in the reliability reporting period).

Once the final average probabilities are created, the scenario generation treats each weather type and incident type as independent. Probabilities for each weather type and incident type for each combination of days (or groups of days) and months (or groups of months) are combined independently to generate the probability of scenarios with no weather or incident effects, scenarios with only weather effects, scenarios with only incident effects, and scenarios with both incident and weather effects.

Modeling of weather events in the computational engine involves modeling each weather event (and incident event if applicable) once in a single run of the study period. Weather events are assumed to occur at either the start of the study period or the middle of it, and they are always modeled for the average duration, unless the duration does not yield a large

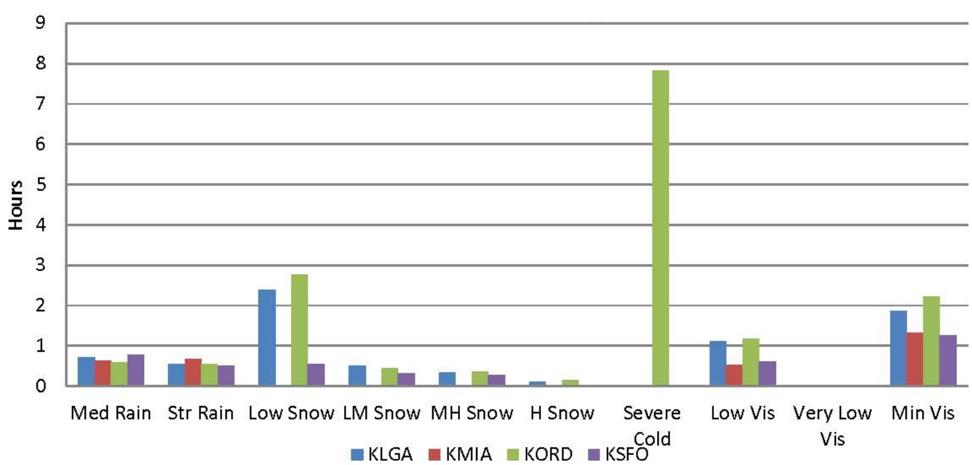

Figure E.5. Average weather event durations in New York, Miami, Chicago, and San Francisco airports, 2001–2010.

enough probability compared with the remainder of the study period with no weather and incident effects.

Weather scenarios modeled in the computational engines are assigned a CAF and free-flow SAF for each weather type for the duration of the modeled weather event taken from the sources mentioned in the introduction.

Weather Event Modeling and Probability Validation

Comparison of Modeled Weather to Historical Weather Events

Four locations in the United States were used to compare the predicted weather events to actual weather events from historical data: Chicago, Miami, New York, New Jersey, and San Francisco. At each location, 10 runs of the Monte Carlo weather event modeling were performed with 10 random seeds. The predicted events were created over 24 h for a full year at 15-min increments using National Climatic Data Center data. The 10-year hourly database (before calculating percentages), in addition to 2011 hourly weather data, was used to identify rain and snow events in historical data.

The Monte Carlo model only predicts a maximum of one storm per day, so the number of storms predicted was compared with total storms, as well as the number of storm days, from 11 years of historical data. Figure E.6 shows the results of the comparison by month with the average, as well as the range of rain storms or rain storm days from both sources for the Chicago metropolitan area. Each month, the total number of storms was significantly underestimated; the number of storm days was better estimated, although it was slightly lower. Exceptions are the winter months, when snow storms were predicted frequently in the place of rain storms. A comparison including snow events shows that the model predicts the frequency fairly accurately other than predicting no April or October snow events, while historical data shows that they occur rarely.

Weather event characteristics were also compared between the modeled weather and historical weather. Figure E.7 shows that the average rain storm intensity (calculated as the total rainfall in an event divided by the event duration) is underestimated in the upper probability section of the cumulative density function. Rain event duration was very accurately modeled in Chicago, and the resulting total rainfall per event was also estimated fairly well despite the underestimated intensities.

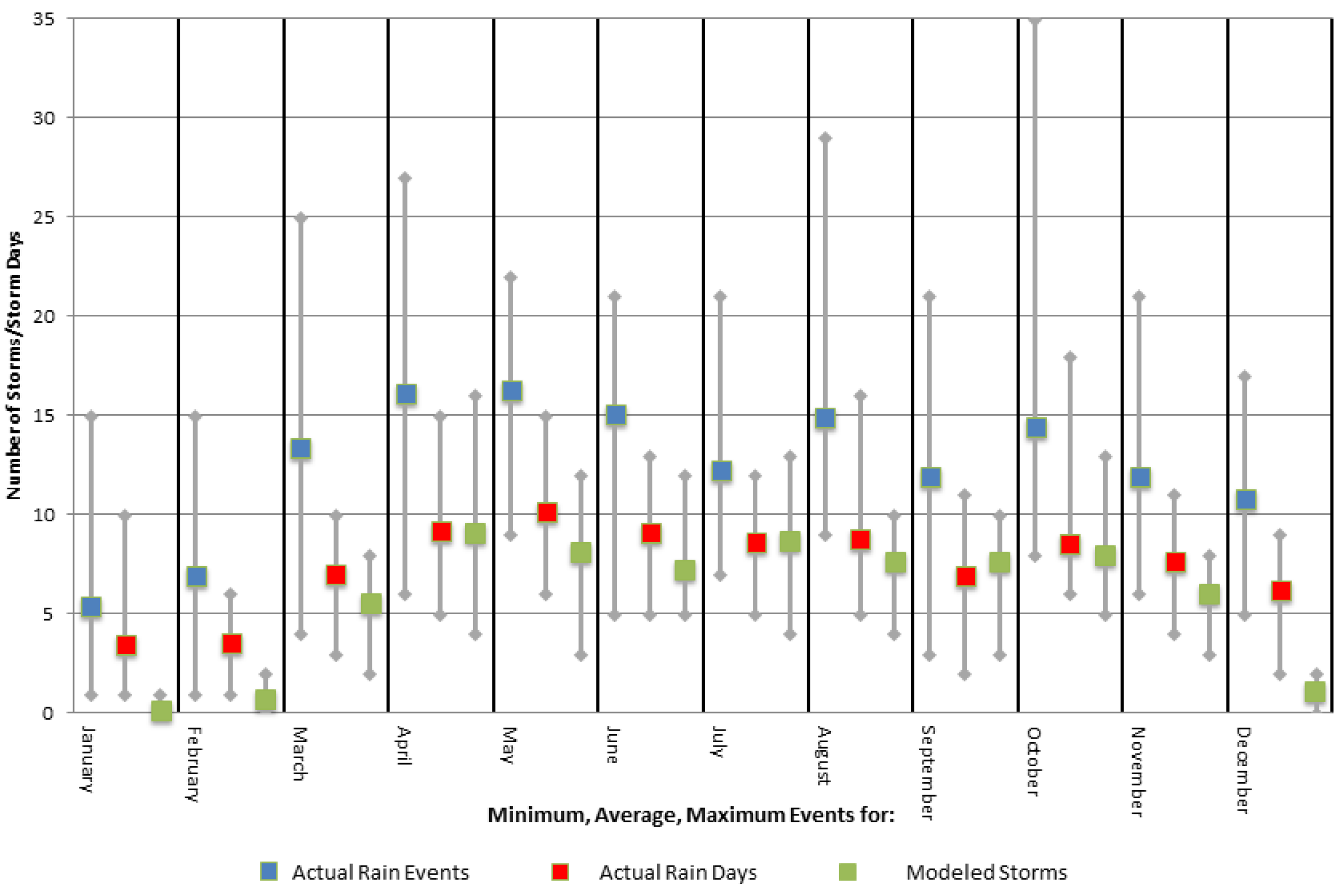

Figure E.6. Chicago airport (KORD) actual versus modeled rain events, 2001–2011 (10 modeled years).

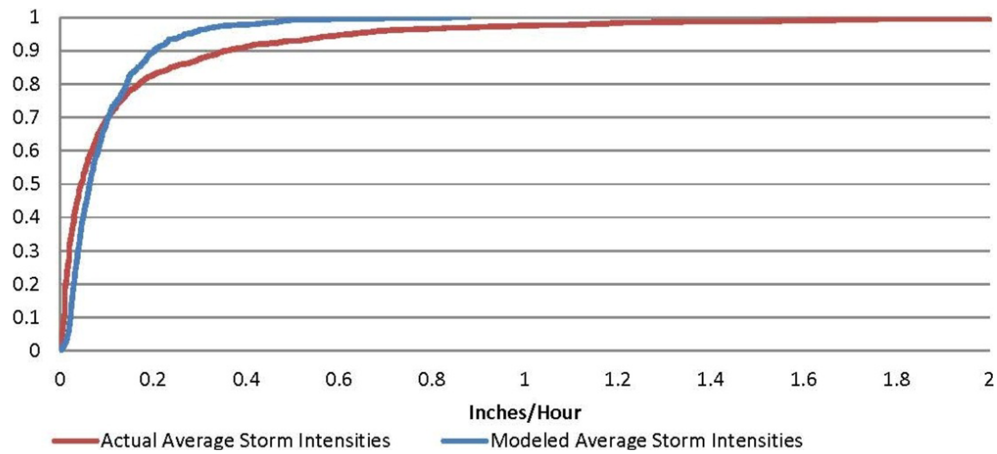

Figure E.7. Chicago airport (KORD) actual versus modeled rain storm intensity, 2001–2011 (10 modeled years).

Although rain event characteristics were fairly well estimated, snow event characteristics had more issues. Average snow event intensities were overestimated, and durations were underestimated. The resulting total precipitation per snow event distribution is shown in Figure E.8, with the modeled events overestimating the total precipitation per event.

Modeled weather events are assigned as rain or snow events based on an underlying distribution of temperature, so the underestimation of rain events in winter and lack of snow events in April and October indicate that this type of temperature–event relationship model has room for improvement. Using the same intensity and duration models for rain and snow caused issues with snow event characteristics; snow events would benefit from a separate model with different underlying distributions.

Probability Confidence Intervals

Weather probabilities can vary greatly year to year, so the reported 10-year averages alone are not good indicators of year-to-year variability. Confidence intervals provide upper and lower bounds to the true average probability; 95% confidence intervals were calculated and graphed along with 2011

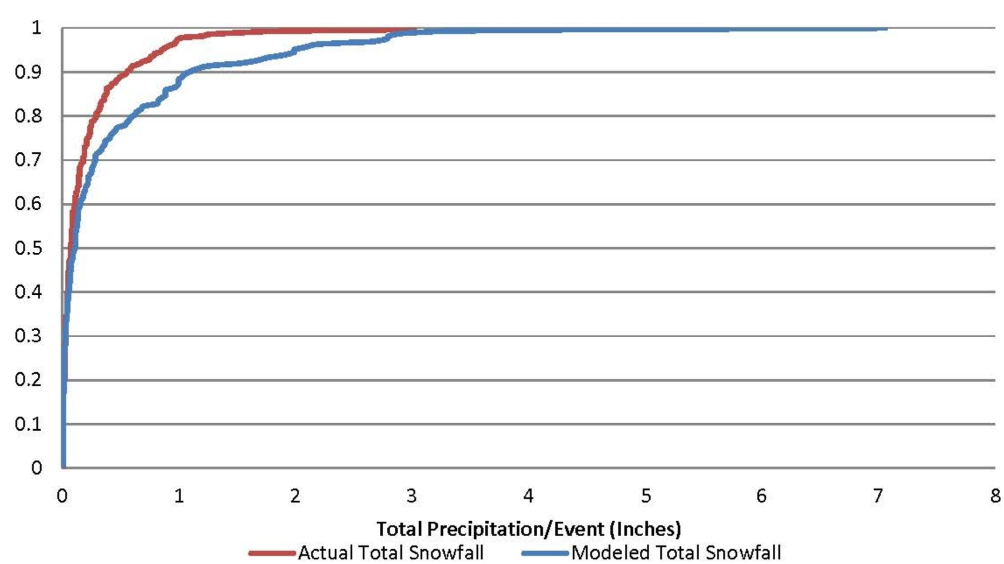

Figure E.8. Chicago airport (KORD) actual versus modeled total precipitation per snow event, 2001–2011 (10 modeled years).

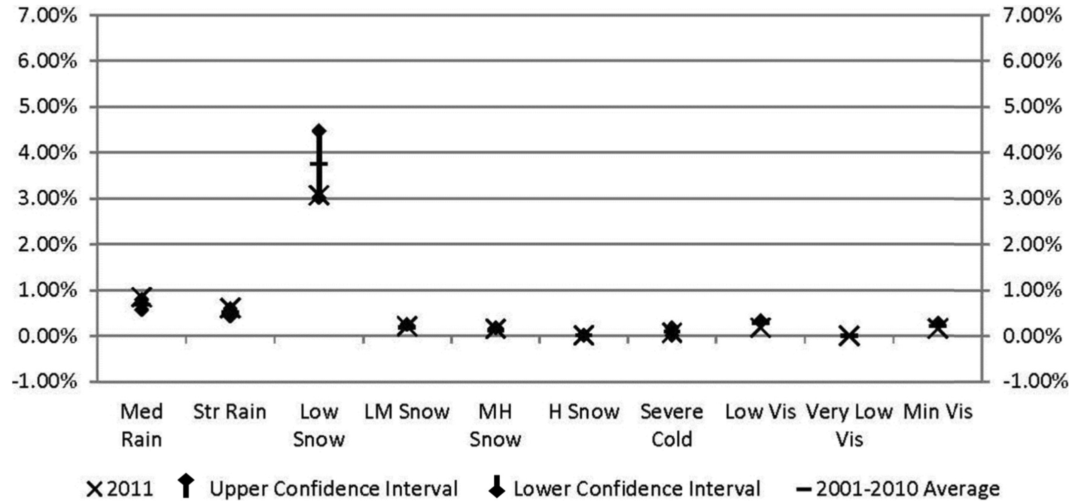

Figure E.9. Chicago annual average weather probability by type, 10-year average versus 2011.

probabilities. For the metropolitan area of Chicago, Figure E.9 shows the annual average probabilities by category; Figure E.10 shows average probabilities for January; and Figure E.11 shows average probabilities for April.

In Chicago, the lowest-intensity snow category occurs most frequently on an annual basis, but the probability confidence interval is also relatively large, indicating high variability. This trend continues when analyzing January and April separately. While confidence intervals are widest for the most frequent weather categories in Figure E.9 and Figure E.10, Figure E.11 shows that the medium rain and low snow categories have very similar averages but very different confidence intervals. Similar month-to-month differences occur across the four locations analyzed.

Estimating Future-Year Probabilities from Historical Averages and Monte Carlo Modeling

As shown in the previous section, annual and monthly weather probabilities contain significant variations from year to year. A sensitivity analysis was performed using the hourly weather data from 2001 to 2011 in Chicago to determine what size sample (or look-back period) is appropriate to estimate

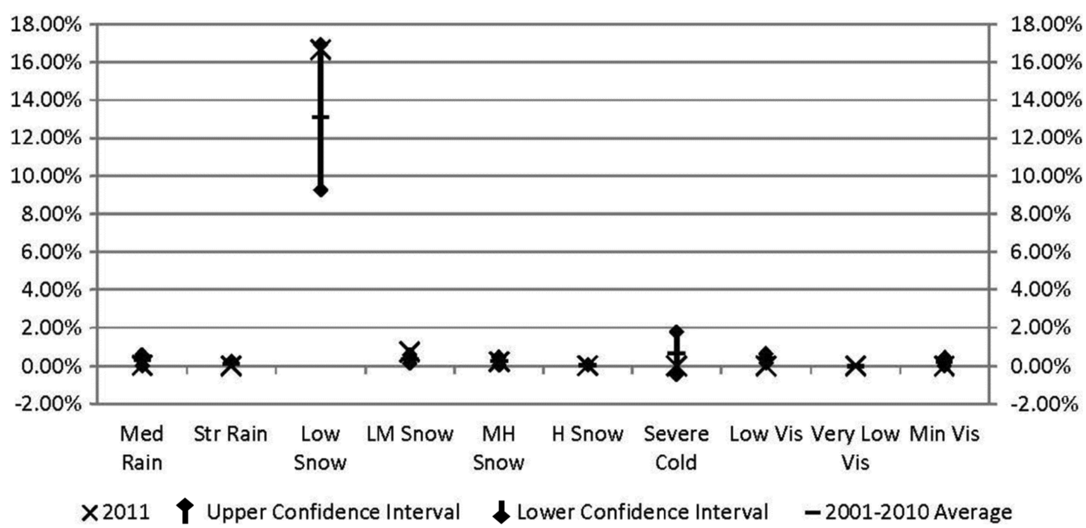

Figure E.10. Chicago average weather probability by type for January, 10-year average versus 2011.

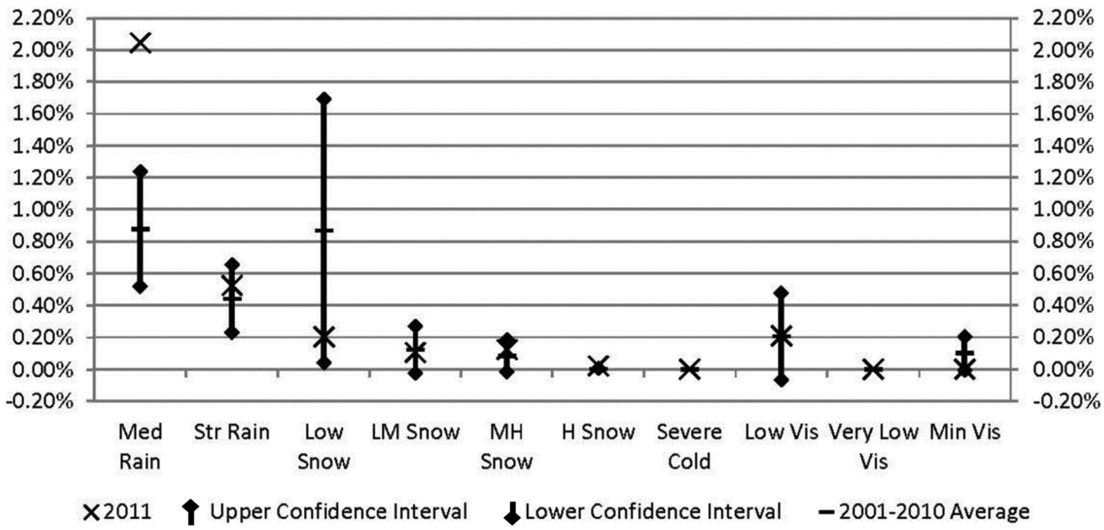

Figure E.11. Chicago average weather probability by type for April, 10-year average versus 2011.

future-year weather probabilities. Both 2011 and 2010 were withheld as estimation years, and average probabilities of the 10 weather categories were calculated on a monthly (12 probabilities per weather category) and annual (one probability per weather category) basis. For 2010, the previous 3-, 5-, 7-, and 9-year average probabilities were compared using root mean square error (RMSE) across all monthly or annual estimates. For 2011, the previous 3-, 5-, 7-, and 10-year averages were compared. Both estimation years were compared with a baseline estimate of 0% probability for each category, as shown in Figure E.12 and Figure E.13. These figures show that the averages of all sample sizes tested—across all weather events—were significantly better than estimating 0% for all monthly or annual weather categories, but they indicate a slight increase in estimation error as the sample size increases for monthly probabilities.

Further analysis indicates that the monthly weather patterns in 2010 were very similar to the previous 3 years, but overall annual estimation error remains very low as the sample size increases. As shown in Figure E.13, the 2011 estimation error trends downward as the sample size increases for both monthly and annual average probabilities.

The Monte Carlo event prediction was also analyzed to create weather probabilities by type for each randomized run of the model with the Chicago weather characteristics. Figure E.14 shows the estimation error for each of the 10 runs when

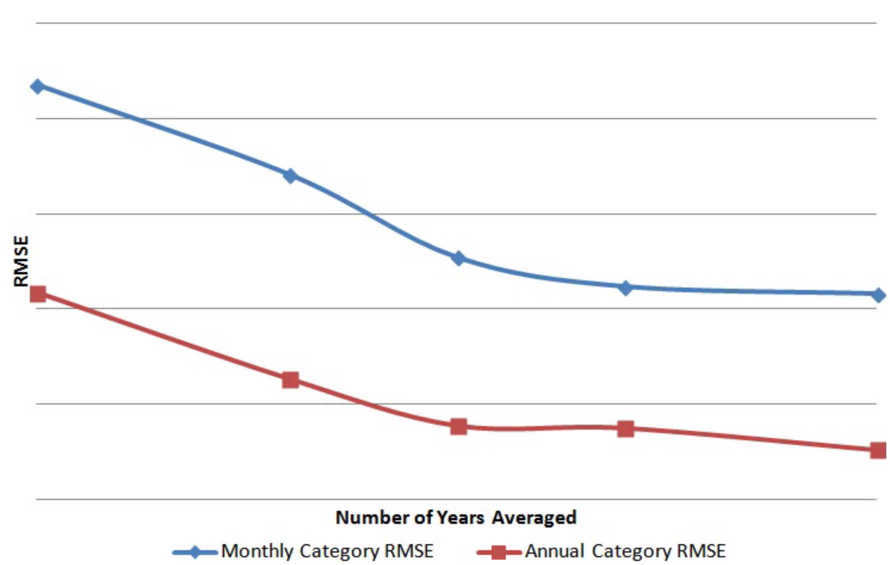

Figure E.12. 2010 Chicago annual weather probability sample size sensitivity.

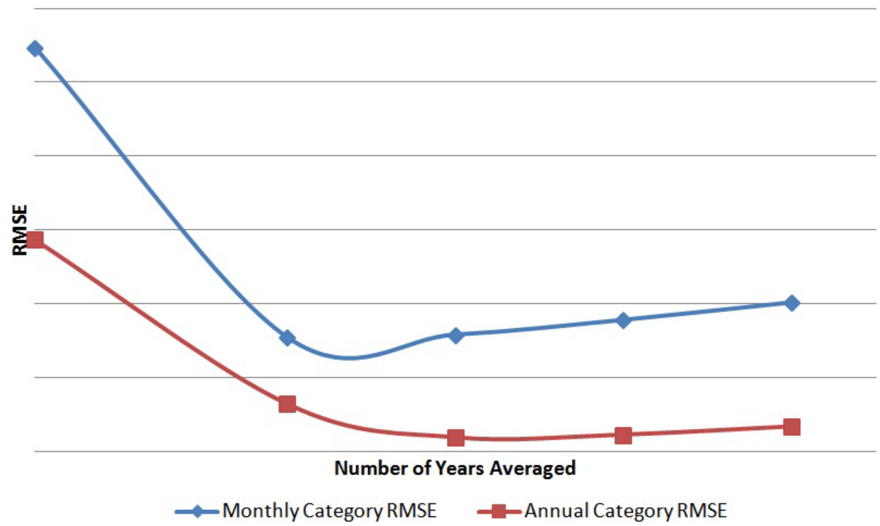

Figure E.13. 2011 Chicago annual weather probability sample size sensitivity.

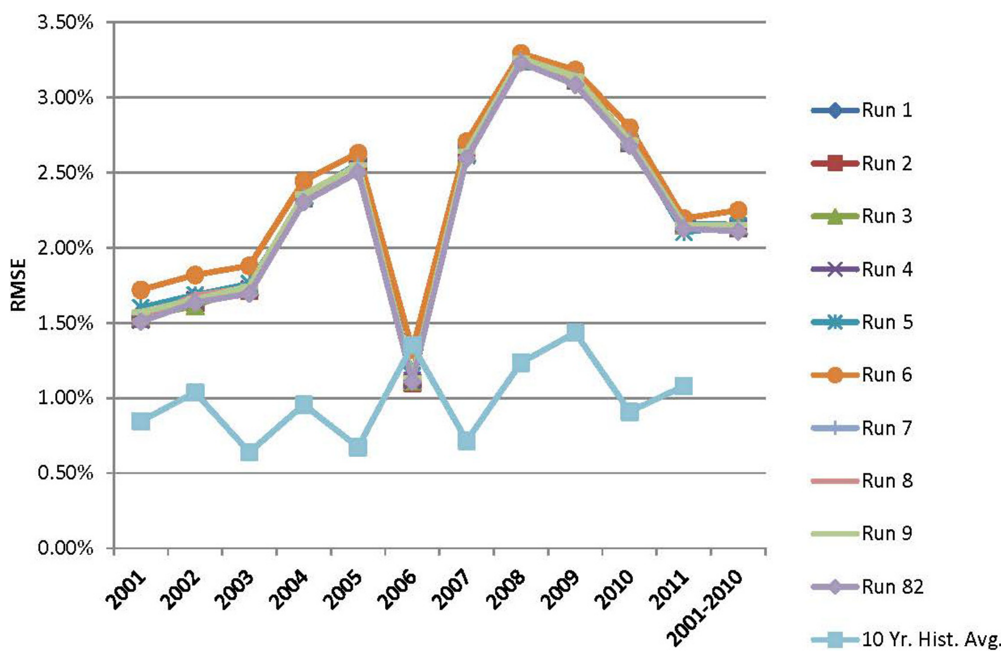

Figure E.14. Monte Carlo predicted probability versus monthly average probability error for Chicago.

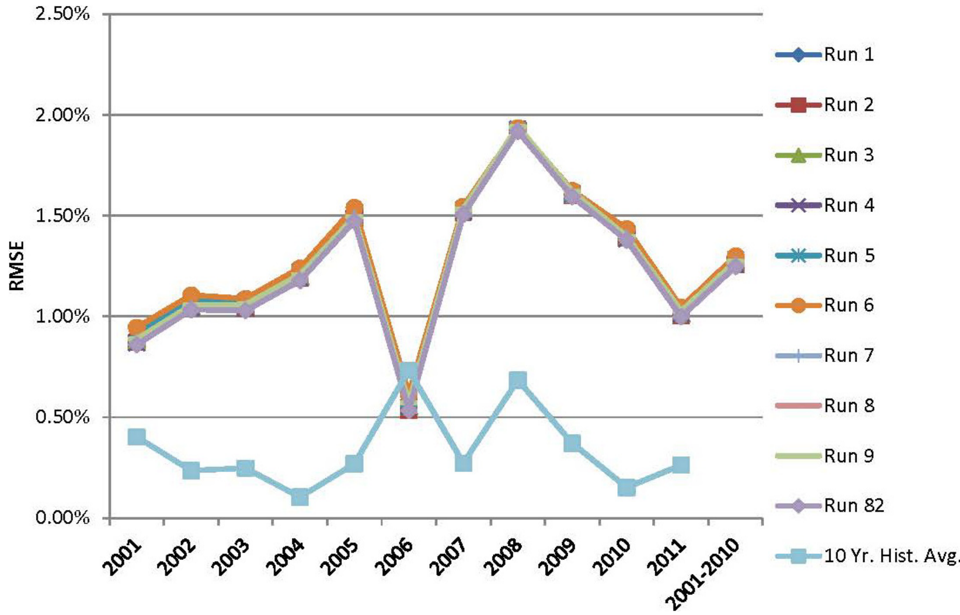

Figure E.15. Monte Carlo predicted probability versus annual average probability error for Chicago.

compared with monthly probabilities of each year of historical data, as well as the 10-year average. Figure E.15 shows the same relationship on an annual level, where error is lower. Of all years, only 2006 is better estimated by the Monte Carlo predictions compared with the 10-year average probability. One source of error for all modeled runs is the limited weather types that can be predicted. Rain and snow events make up only five of the 10 weather types used by the freeway scenario generator; however, total error increases when only including those five categories compared with assuming the other categories to have a probability of 0%. Otherwise, error within the categories that are able to be modeled can be attributed to either climatological data informing the model or underlying distributions used in the model.

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APPENDIX F

Incident Probabilities Estimation for Freeway Scenario Generator

Incidents Defined and Classified

An incident is an unplanned disruption to the capacity of the facility. Incidents do not need to block a travel lane to disrupt the capacity of the facility. They can be a simple distraction within the vehicle (e.g., spilling coffee) or on the side of the road or the reverse direction of the facility.

Incidents can be classified according to the response resources and procedures required to clear the incident. This classification helps in identifying strategic options for improving incident management.

However, the *2010 Highway Capacity Manual* (HCM2010) (Transportation Research Board of the National Academies 2010) classifies incidents on freeways only by number of lanes blocked, and if the incident is on the shoulder, by whether it is a collision or not. The HCM does not deal with incidents on urban streets.

Section 6I.01 of the *Manual on Uniform Traffic Control Devices* (Federal Highway Administration 2009) classifies incidents according to their expected duration:

- Extended duration incidents are expected to persist for over 24 h and should be treated like work zones.
- Major incidents have expected durations of over 2 h.
- Intermediate incidents have expected durations of 0.5 h up to and including 2 h.
- Minor incidents are expected to persist for less than 30 min.

Scenario Generation Fundamentals

Estimating the probabilities of incidents is a key element in freeway travel time reliability analyses. Incident probabilities feed into the base-scenario generation procedure in order to enumerate and characterize the full variability of the operational status of the freeway facility. Once these operational statuses are analyzed, the estimated travel time distribution can be generated. The estimated travel time distribution, in turn, provides the primary basis for reliability performance assessment.

In the SHRP 2 Project L08 analysis, three basic stochastic events affect travel time: variability in demand, weather, and incidents (Rouphail et al. 2012). The goal of this section is to stochastically model incidents on a freeway facility and characterize the properties that are capable of providing the required information for the reliability performance assessment. Specifically, this section provides the methodology for probability estimation of incident scenarios. Incidents are stochastic in nature in that their occurrence and duration are probabilistic with certain distributions and parameters. The location and start time are two other stochastic dimensions of incident occurrence.

The freeway scenario generator (FSG) provides the analyst with multiple paths to estimate the required incident occurrence probabilities. Analysts have the option of directly entering monthly incident probabilities, as well as the option of using national default values to generate the probabilities. In either case, the incident probability for a given time period is the fraction of the overall analysis time period or reliability reporting period that a specific incident type occurs on the facility. This section uses the incident attribute *type* to address the severity in terms of lateral lane closure. The process flow and theoretical background behind FSG are discussed later in this section. The default values provided in the FSG are based on national data for incidents and crashes from nine freeway facilities. These default values are provided for the analyst to use in the absence of local data.

Appropriate mathematical models are required to account for the stochastic behavior of incidents on freeway facilities. The proposed mathematical model is a queuing theory approach that models the freeway as a queuing system in which incidents represent the service provided for vehicles. The incident occurrence rate and its duration are fed into the model to fully characterize the problem.

In this study, the I-40 freeway in North Carolina was selected to demonstrate the application of the FSG in estimating monthly incident probabilities. A 12.5-mi section of the facility was analyzed in the Research Triangle Park area near Raleigh. Incident data logs (albeit incomplete) were made available to the research team from the North Carolina Department of Transportation (NCDOT) for the year 2010. The estimated probabilities based on the developed mathematical models were compared with incident data extracted from the logs.

The FSG uses a deterministic approach for creating and characterizing the operational scenarios for a specific facility in a specified reliability reporting period. This deterministic methodology categorizes factors that affect travel time distribution. An important limitation of the methodology is the assumption of independence between the various contributing factors. This assumption simplifies the estimation of joint probabilities by simply multiplying the individual factor probabilities. Each unique combination of demand level, weather effect, and incident type is termed a base scenario in the FSG.

The incorporation of incident impacts includes one additional step, namely modeling the incident effect within the study period, using the HCM freeway facilities model FREEVAL to estimate the resulting travel time. Incident modeling is done by inserting appropriate capacity adjustment factors (CAFs), free-flow speed adjustment factors (SAFs), and the number of open lanes associated with the incident. This section focuses on incident probability estimation. Table F.1 presents the incident probability table in the FSG used to generate the base scenarios. For example, the probability of having a two-lane closure incident in April is 1.94%. This means that if a study period in April were selected at random, about 1.94% of the duration of that study period (on average) would involve a two-lane closure incident.

There are other ways to categorize incidents on freeway facilities. FSG uses the categorization based on incident severity (lane or shoulder closure) and calls those incident types. More detailed information and categorizations are provided in the project team's incident white paper. The categories used in the FSG methodology are no incident, shoulder closure, one-lane closure, two-lane closure, three-lane closure, and four-lane closure. It is assumed that this categorization best describes different impacts of incidents on the facility travel time distribution.

Data Requirements

Implementing agencies that have access to high-quality incident-log data are in a data-rich environment. By high quality, the authors mean precise designation of the sequence of events around the incident, including start time, duration, number of lane closures, clearance time, mileposts affected, and so forth. If such data are not available (which is the majority of cases), then the analyst is advised to use national default values along with certain mathematical models for estimating study period monthly incident probabilities. The second condition is called a data-poor or semi-data-poor environment.

Estimating Incident Probabilities in a Data-Rich Environment

In data-rich environments, agencies can directly estimate the probabilities for different incident types by analyzing their entries in the incident logs. The data logs should have the

Table F.1. Monthly Probabilities for Different Incident Types: I-40 Eastbound

	Probability of Different Incident Types					
Month	No Incident	Shoulder Closure	One Lane Closure	Two Lane Closure	Three Lane Closure	Four Lane Closure
January	66.42%	23.30%	7.06%	1.79%	1.43%	0.00%
February	66.36%	23.34%	7.08%	1.79%	1.43%	0.00%
March	65.10%	24.18%	7.36%	1.87%	1.49%	0.00%
April	63.79%	25.05%	7.66%	**1.94%**	1.56%	0.00%
May	63.87%	25.00%	7.64%	1.94%	1.55%	0.00%
June	64.53%	24.56%	7.49%	1.90%	1.52%	0.00%
July	64.10%	24.85%	7.59%	1.93%	1.54%	0.00%
August	65.30%	24.04%	7.32%	1.86%	1.48%	0.00%
September	65.97%	23.60%	7.17%	1.82%	1.45%	0.00%
October	65.04%	24.22%	7.38%	1.87%	1.50%	0.00%
November	66.79%	23.05%	6.98%	1.77%	1.41%	0.00%
December	68.56%	21.86%	6.59%	1.67%	1.33%	0.00%

incidents recorded and categorized as defined in a previous section of this appendix, along with their durations. Study period monthly probabilities of different incident types are computed from Equation F.1. The study period represents the number of hours in a given scenario in which the facility's operations are analyzed.

$$\text{Prob}\{\text{incident type } i \text{ in a SP in month } j\} = \frac{\text{Sum of minutes in all SPs in month } j \text{ that incident type } i \text{ is present}}{\text{Sum of all SP minutes in month } j} \quad \text{(F.1)}$$

where SP is study period.

Equation F.1 gives the fraction of time that incident type i is present on the facility, which is equal to the probability of having an incident at any time instance during the study periods in month j. Specifically, the probabilities required in the FSG are the time-wise probabilities of the presence of certain incident conditions. Conversely, the probabilities do not indicate the frequency or the chance of occurrence of an incident.

In a data-rich environment, the monthly incident probabilities computed from data logs are inserted in the FSG directly. The quality of data is paramount in this case as there could be some cases of unreported incidents, which could introduce bias and errors into the travel time distribution.

The example illustrated in this appendix pertains to I-40 eastbound (EB) in Raleigh in 2010. NCDOT compiled incident data logs for I-40 based on reported incidents. The Traffic Management Center is in charge of incident data gathering based on phone calls and reports for incidents. According to the incident data logs, the incident's causes are not clear in the majority of cases. However, based on sensor readings from traffic.com, some accurate information is inserted into the database, such as speed of vehicles at the incident time.

The I-40 facility incident database contains several attributes: freeway name, mile marker, county, city name, start time, end time, and reason/description. The first step is to select the appropriate records from this database. For this purpose, all incidents from Mile Marker 279 to 293 of I-40 were filtered and extracted. In 2010, according to crash reports, approximately 90 collisions occurred on the facility; however, the incident log maintained by North Carolina's Transportation Information Management System reported only 32 collisions. During the peak period, 142 incidents were reported, but 29 of those reports stated "Traffic traveling at 0 mph at X" location instead of reporting a lane blockage or the actual location of the incident. The remaining reports were the only ones reporting lane blockages, and of those, only four shoulder incidents were reported.

The distribution of incidents by lane blockage shown in Table F.2 is significantly different from the national urban defaults. The incidents reported do not have enough information to be modeled at the level of detail required by the FREEVAL model. The research team thus concluded that data in the incident logs were not of sufficient quality for the purpose of this study, given that the generated incident probabilities are uncharacteristically low. This led the SHRP 2 L08 project research team to use a semi-data-poor approach for generating the incident probabilities for the I-40 EB case study.

Estimating Incident Probabilities in Data-Poor or Semi-Data Poor Environments

Figure F.1 shows the data structure proposed for the predictive methodology. The data required in each purple and green box in Figure F.1 can be estimated based on available local data or substituted with national default values.

Table F.2. Incident Probabilities for I-40 EB Based on Analysis of Data Logs

	Probability of Different Incident Types					
Month	**No Incident**	**Shoulder Closure**	**One Lane Closure**	**Two Lane Closure**	**Three Lane Closure**	**Four Lane Closure**
January	92.90%	5.00%	1.50%	0.50%	0.10%	0.00%
February	94.50%	4.00%	1.00%	0.40%	0.10%	0.00%
March	94.60%	4.00%	1.00%	0.40%	0.00%	0.00%
April	96.20%	3.00%	0.50%	0.30%	0.00%	0.00%
May	97.20%	2.00%	0.50%	0.30%	0.00%	0.00%
June	98.65%	1.00%	0.25%	0.10%	0.00%	0.00%
July	98.65%	1.00%	0.25%	0.10%	0.00%	0.00%
August	98.65%	1.00%	0.25%	0.10%	0.00%	0.00%
September	97.20%	2.00%	0.50%	0.30%	0.00%	0.00%
October	97.20%	2.00%	0.50%	0.30%	0.00%	0.00%
November	95.60%	3.00%	1.00%	0.40%	0.00%	0.00%
December	93.40%	4.00%	2.00%	0.50%	0.10%	0.00%

Figure F.1. Proposed data structure for estimating incident probabilities in a data-poor environment.

Incident Rates Estimation in the Study Periods in a Month

All subsequent discussions in this section refer to items numbered 1, 2, and 4 in Figure F.1. Two possible approaches for incident rate estimation can be carried out. The first is by directly estimating the rates from the incident data logs. Alternatively, one can use the prevailing crash rate and apply a local or default crash-to-incident (CTI) factor to compute the incident rate. In the project team's incident white paper, the national default value for the CTI rate is 4.9. In the FSG the analyst may also specify a local CTI. Table F.3 presents CTI factors for freeway facilities. For the I-40 EB case study, and based on a study by Khattak and Rouphail (2005), CTI is estimated at 7.2. In the current FSG implementation a rounded value of 7.0 has been entered.

Factors in this table were developed based on data sets from Washington, Virginia, Florida, Georgia, Maryland, and California for freeways and from Oregon, California, and Illinois for arterials.

If the local crash rate is unavailable, or for future scenario analyses (Use Case 2), the HERS model (Federal Highway Administration 2005) is used in the FSG to estimate monthly crash rates on the facility. Agencies may use other predictive models, such as those in the *Highway Safety Manual* (American Association of State Highway and Transportation Officials 2010), to estimate the monthly crash rate for the facility. The crash or incident rate is estimated per 100 million vehicle miles traveled (VMT). The HERS model uses Equation F.2 to estimate the crash rate per 100 million VMT based on a few facility attributes:

$$\mathrm{CR} = \begin{pmatrix} 154.0 - 1.203 \times \mathrm{ACR} + 0.258 \times \mathrm{ACR}^2 \\ - 0.00000524 \times \mathrm{ACR}^5 \end{pmatrix} \times \exp(0.0082 \times [12 - \mathrm{LW}]) \tag{F.2}$$

where

CR = crash rate;
ACR = facility annual average daily traffic (AADT) divided by two-way hourly capacity; and
LW = lane width, ft.

ACR is estimated to be 7.9 h per lane for the I-40 EB case study, which yields a crash rate equal to 159 crashes per 100 million VMT. The L08 project research team decided to use its own estimate, starting with a local crash rate of 164.5 for urban freeways in Wake County, North Carolina, where the facility resides.

Table F.3. Crash-to-Incident (CTI) Factors for Freeway Facilities

Statistic	CTI Factor
Range	2.4–15.4
Average	4.9
Median	6.5

Incident Severity

This section demonstrates the procedure related to Item 3 (incident severity distribution) in Figure F.1. The distribution of incident severity must be known a priori for incorporation in the methodology. This distribution is defined by $\mathbb{G}(i)$, which is assumed to be homogeneous across the facility and demand levels. Agencies can estimate this distribution by analyzing the incident logs or by using the national default values provided

Table F.4. National Default Distribution of Incident Severity and Duration

Severity	Type of Closure			
	Shoulder	Single Lane	Two Lanes	Three+ Lanes
Mean and probability range	75.4% (28.8–96.3)	19.6% (3.0–65.6)	3.1% (0.5–7.5)	1.9% (0.6–4.3)
Mean[a] and duration range (min)	34.0 (8.7–58.0)	34.6 (16.0–58.2)	53.6 (30.5–66.9)	69.6 (36.0–93.3)

[a]Durations are discretized to the nearest 15 min later on. Values are based on data from Washington, Virginia, Florida, Georgia, Maryland, and California.

in Appendix D and depicted in Table F.4. Equation F.3 gives a definition of $\mathbb{G}(i)$ as a discrete distribution, where i denotes the incident type (e.g., $i = 1$ is equal to shoulder closure and $i = 5$ is a four-lane closure).

$$\mathbb{G}(i) = \begin{cases} g_1 & z = 1 \\ g_2 & z = 2 \\ g_3 & z = 3 \\ g_4 & z = 4 \\ g_5 & z = 5 \end{cases} \tag{F.3}$$

Incident Mean Duration

The incident mean duration is naturally required to estimate monthly incidents probabilities based on the formulation in Equation F.1. Although the duration of incidents are probabilistic and generally follow a lognormal distribution, in the proposed methodology only the mean value of durations is required for each incident type. Agencies could either enter local mean duration values for the facility or use the national default duration values provided in Table F.4.

The duration of incidents is denoted by $\mathbb{D}(i)$, and the mean duration of an incident with severity i is expressed by $E[\mathbb{D}(i)]$, where i represents the severity of the incident. $\mathbb{D}(i)$ follows a discrete distribution of mean duration of incidents. For the I-40 case study, national default durations were incorporated in evaluating monthly incident probabilities.

Demand Level

The proposed methodology requires the demand level to be known in order to estimate the number and probability of incidents. Specifically, the crash or incident rate is estimated per 100 million VMT, which means that to yield the expected number of incidents, demand and facility length must be known. The demand is characterized based on AADT across the facility, along with a set of demand multipliers that vary the demand levels across the week and between months.

In a data-rich environment in which agencies have access to demand data for a specific study period (also called a seed file), VMT can be directly calculated for each 15-min period. Adding these 15-min VMTs yields total VMT for the study period. Demand multipliers are used to adjust the demand for each demand pattern based on the seed file data. Thus, the VMT is divided by the demand multiplier associated with the seed file's date. Further dividing the resulting VMT by the fraction of demand in the study period ($\sum_{t \in SP} k_t$) produces a good estimate of the directional AADT for the facility.

The main reason for estimating the AADT based on VMT is that the demand distribution across the facility is not necessarily uniform during each time period, since some ramps could have a different demand distribution (over time) compared with the mainline entry AADT. Equation F.4 shows the relationship between study period VMT, segment length, and demand on each facility segment:

$$\text{VMT} = \frac{\sum_{t \in \text{SP}} \left(\sum_{k \in \text{Facility Segments}} L_k \times D_k^t \right)}{4} \tag{F.4}$$

where L_k represents the length of segment k, and D_k^t is the hourly demand on segment k in time period t.

Equation F.5 shows how directional AADT can be computed when a demand seed file is available:

Directional Average AADT across the facility = DAADT

$$= \frac{\text{VMT}}{\text{DM}_{\text{Seed}} \times \left(\sum_{t \in \text{SP}} K_t \right) \times L_f} \tag{F.5}$$

where ($\sum_{t \in SP} K_t$) represents the portion of daily demand occurring during the study period, and DM_{Seed} is the demand multiplier associated with the seed file. In cases for which only overall AADT data are available, the directional distribution along with the segments length is used to estimate the directional facility AADT according to Equation F.6:

Directional AADT across the facility = DAADT

$$= \frac{\left(\sum_{k \in \text{Facility Segments}} L_k \left(\text{DAADT}_k \right) \right)}{\left(\sum_{k \in \text{Facility Segments}} L_k \right)} \tag{F.6}$$

where DAADT_k is the directional AADT on segment k.

Core Incident Methodology Model

This model begins with the widely used assumption that the number of incidents in a given study period is Poisson distributed (Skabardonis et al. 1997). The first step is to compute the expected number of incidents on the freeway facility during the study period by using Equation F.7:

$$n_j = \text{IR}_j \times \text{DAADT} \times \text{DM}_j \times k_s \times L_f \quad \text{(F.7)}$$

where

n_j = expected number of incidents in a study period (only) in month j;
IR_j = incident rate per 100 million VMT in month j;
DM_j = weighted average demand multiplier for month j;
k_s = fraction of daily demand that occurs during the study period; and
L_f = length of facility (mi).

Note that DAADT is the directional AADT and could be substituted by AADT × DD, where DD is directional distribution. IR_j is either known or is estimated from Equation F.8 from CR_j, which is the crash rate in month j:

$$\text{IR}_j = \text{CR}_j \times \text{CTI} \quad \text{(F.8)}$$

M/G/∞ Queuing Modeling of Incidents on a Freeway Facility

A stochastic queuing model is used for modeling incidents in the freeway facility. Table F.5 presents the definition of the queuing model used for computing the probability of incidents.

The arrival of vehicles involved in an incident follows the Poisson distribution, because the incident occurrence follows a Poisson distribution with mean n_j in month j. The probability of having x incidents in the SP in month j is computed from Equation F.9. In the queuing systems, this arrival distribution is denoted by M (Markovian).

$$\text{P}_j(x) = \frac{n_j^x}{x!} e^{-n_j} \quad \text{(F.9)}$$

where $P_j(x)$ is the probability of having x incidents in a study period in month j, and n_j is the expected number of incidents in the study period for the same month. The service rate has an unknown distribution, where in the queuing systems it is denoted by G (general distribution). Since any lane of any segment of the freeway is operating as a server in the queuing system, the number of servers is infinite in this queuing model. Thus, the queuing model used for modeling incidents in the freeway facility is $M/G/\infty$.

For the queuing model considered, the probability of having no incident on the freeway facility is first calculated, and then subtracted from one to yield the incident probability. Because there could be more than one incident, the team recommends the use of complementary probabilities. Equation F.10 shows this computation:

$$\begin{aligned}&\text{Prob}\{\text{having one or more type } i \text{ incident}\}\\ &= 1 - \text{Prob}\{\text{having no incidents}\}\\ &= 1 - \text{Prob}\{\text{having no customers in system}\} = 1 - P_0 \quad \text{(F.10)}\end{aligned}$$

Based on Equation F.11, which is founded on the steady state (long-run) probability of having no customers in the queuing system, the probability of having no incident in the freeway facility is defined (Adan and Resting 2001):

$$P_0 = e^{-(n_j \times g_i \times E[\mathbb{D}(i)])} \quad \text{(F.11)}$$

Because the probability of each incident type needs to be estimated separately in the FSG, the incident occurrence rate should multiplied by g_i, because the occurrence of different incident types is independent, and g_i is considered homogeneous in time. The occurrence of each incident type

Table F.5. Definition of Incident's Queuing Model for a Freeway Facility

Queuing System Concept	Transportation Equivalent	Description
Arrival rate	Incident's occurrence rate	n_j is the arrival rate per SP in month j.
Service rate	Incident's clearance rate	$\frac{1}{E[\mathbb{D}(i)]}$ is the service rate per SP for incident type i.
Service duration	Incident's duration	$E[\mathbb{D}(i)]$ is the incident's mean duration; the unit is the SP.
Server	Any lane of any segment of the facility	There are infinite such servers where the incident can occur on the facility.

Table F.6. Description of Process Flowchart

Step	Label	Description
1	Are incident data logs available for the facility?	If an agency has no access to incident data logs, select NO; otherwise select YES.
2	Is quality of data excellent?	This step assesses the quality of incident-log data; if minimum quality thresholds are not met, select NO; otherwise select YES.
3	Use Equation F.1 to compute the monthly incident probabilities.	Directly estimate the monthly incident probabilities. Equation F.1 can be used for this purpose.
4	Is incident rate available?	This step attempts to estimate incident rates based on local data. If those data are available, select YES; otherwise select NO.
5	Use Figure F.3 process for estimating incident rate.	Instructions to estimate incident rate are provided in the flowchart in Figure F.3.
5.1	Is crash rate available?	If local crash rate (per 100 million VMT) for the facility is available, select YES; otherwise select NO.
5.2	Use HERS model to estimate the crash rate.	If local crash rate is unavailable, or for future-year predictions, HERS model is recommended to estimate crash rate for the facility.
5.3	Is crash-to-incident rate available?	If CTI factor for the facility is known, select YES; otherwise select NO.
5.4	Use default crash-to-incident rate.	CTI = 4.9 should be used in the analysis.
5.5	Use Equation F.8 to compute the incident rate per study period in month.	Equation F.8 is used to compute the incident rate.
6	Is incidents severity distribution known?	If incident type distribution for facility is known, select YES; otherwise select NO.
7	Use national default values.	Use national defaults for distribution of incident types in incident paper.
8	Is duration of incident types known?	If mean duration for each incident type is known, select YES; otherwise select NO.
9	Use national default values.	Use national defaults for the mean duration of each incident types in incident paper.
10	Using core computational procedure, estimate the monthly incident probabilities per study period.	Equations F.7 through F.13 are used to estimate study period incident probabilities for each month.

is also Poisson with the rate $n_j \times \mathcal{g}_i$. Thus, the probability of incident type i in month j is computed using Equation F.12:

$$\text{Prob}\{\text{incident type } i \text{ in an SP in month } j\} = 1 - P_0$$
$$= 1 - e^{-(n_j \times \mathcal{g}_i \times E[\mathbb{D}(i)])} \tag{F.12}$$

Thus, the probability of having any incident is set to be equal to the probability of having at least one incident in the system. Equation F.13 is thus the estimation alternative to Equation F.12 when data are not available.

The probability of having no incident in the system is equal to one minus the probability of having any incident type (i), with $i = 1, 2, \ldots, 5$.

$$\text{Prob}\{\text{No incident in the system}\}$$
$$= 1 - \text{Prob}\{\text{Any type of incident}\}$$
$$= 1 - \sum_{i \in \text{incident types}} 1 - e^{-(n_j \times \mathcal{g}_i \times E[\mathbb{D}(i)])} \tag{F.13}$$

Recommended Process Flow

Since there could be many paths for agencies to compute the incident probabilities, this section presents a process flow that is compatible with the theoretical aspects of the paper. Table F.6 describes the process flow steps depicted in Figure F.2 and Figure F.3. Figures F.2 and F.3 present the flowchart of the process flow for agencies to follow while completing the Incidents worksheet in the FSG.

Numerical Example for I-40 EB Case Study

In this section, a numerical example is provided to demonstrate the detailed calculations executed in the FSG to estimate the incident probabilities per study period in any given month. This example focuses on the probability of a one-lane closure incident to occur in a study period in May 2010 for the I-40 EB case study. The local crash rate used for I-40 EB is 164.5 crashes per

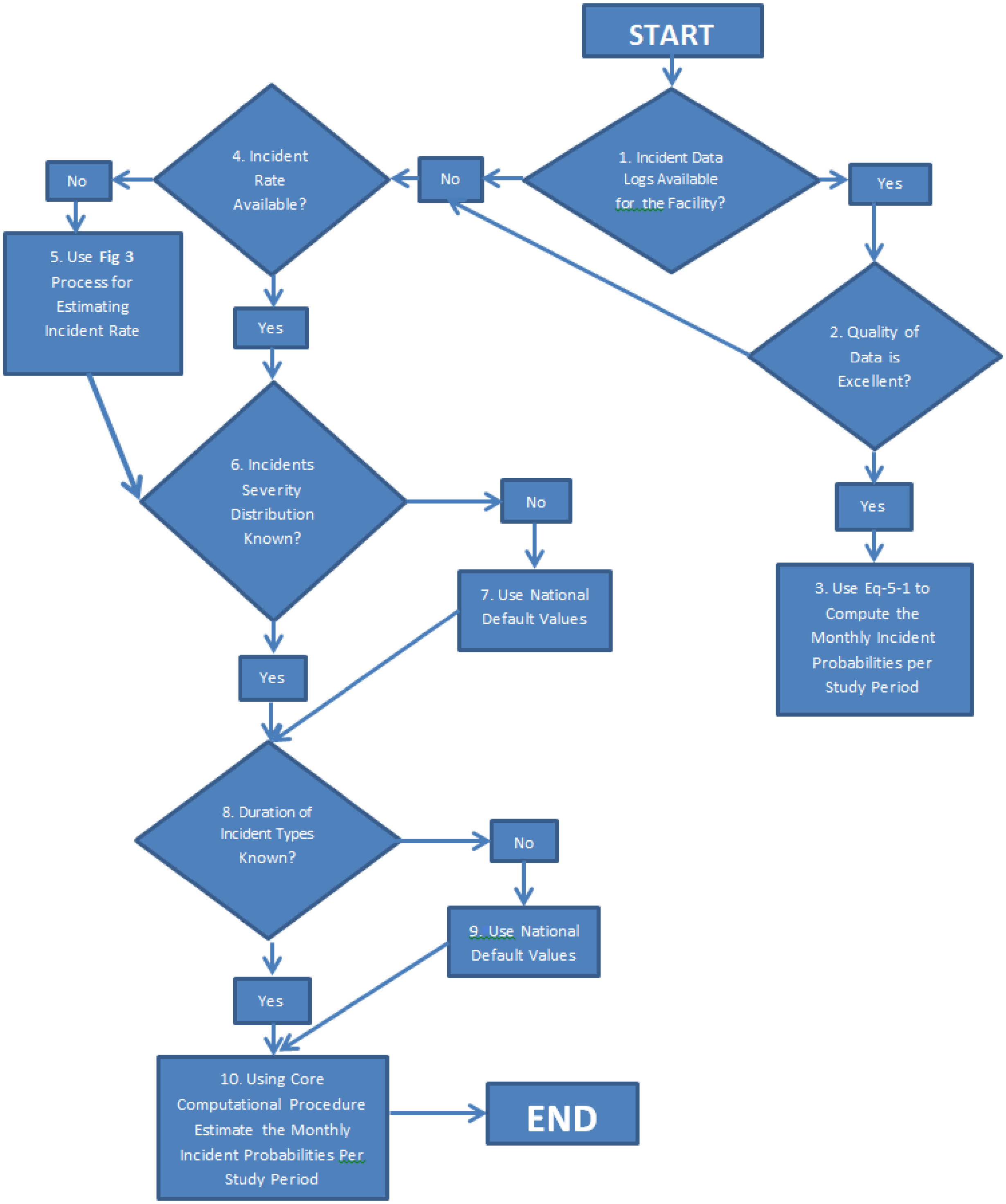


Figure F.2. Proposed process flow.

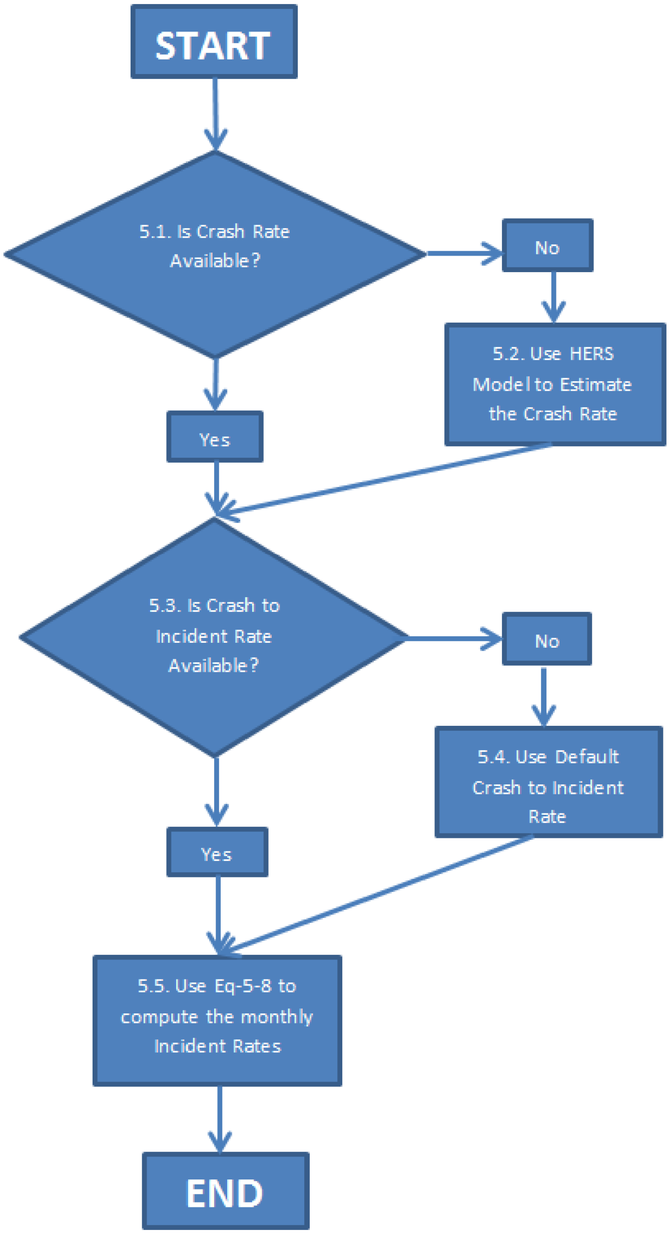


Figure F.3. Proposed process flow for incident rate estimation.

100 million VMT, and the CTI factor is 7 based on local data (Khattak and Rouphail 2005). Table F.7 shows the schematic of the incident data input file for the FSG with this specified configuration. The study period duration is 6 h, or 360 min.

Table F.8 shows the demand multipliers for I-40 EB for all weekdays in May. The demand multiplier (DM) for May is calculated based on Equation F.14.

$$\mathrm{DM}_j = \frac{\sum_{\text{for each month}} \begin{pmatrix} \text{Number of days in each month} \\ \text{with demand pattern } j \end{pmatrix} \times (\mathrm{DM})}{\text{Total number of days in demand pattern } j} \tag{F.14}$$

Based on Equation F.14, DM_5 (i.e., the demand modifier for May, the fifth month) is calculated at 1.13 as shown below:

$$\mathrm{DM}_5 = \frac{5\times1.076+4\times1.106+4\times1.114+4\times1.158+4\times1.210}{21}$$

$$= 1.13$$

The overall incident rate is computed from Equation F.15:

$$\mathrm{IR}_j = \mathrm{CR}_j \times \mathrm{CTI} = 164.5 \times 7$$

$$= 1{,}151.5 \text{ per 100 million VMT} \tag{F.15}$$

The 6-h study period overall directional demand factor is 0.3884, and the facility length is 12.5 mi. The average AADT for I-40 is estimated from Equation F.16 and Equation F.17.

$$\mathrm{VMT} = \sum_{t\in \mathrm{SP}} \frac{\left(\sum_{k\in \text{Facility Segments}} L_k \times D_k^t\right)}{4}$$

$$= 330{,}006 \text{ vehicle miles} \tag{F.16}$$

Directional AADT across the facility = DAADT

$$= \frac{\mathrm{VMT}}{\mathrm{DM}_{\text{Seed}}\left(\sum_{t\in\mathrm{SP}} K_t\right) \times L_f}$$

$$= \frac{330{,}006}{1\times0.3884\times12.5} = 67{,}972 \text{ veh} \tag{F.17}$$

The frequency of incidents in the study period is computed from Equation F.18:

$$n_5 = \mathrm{IR}_5 \times \mathrm{AADT} \times \mathrm{DD} \times \mathrm{DM}_5 \times k_s \times L_f$$

$$= 1{,}151.5 \times 10^{-8} \times 67{,}972 \times 1.13 \times 0.3884 \times 12.5$$

$$= 4.294 \text{ incidents per study period in May} \tag{F.18}$$

Using the duration of a one-lane closure incident (34 min based on national default data), then $E[\mathbb{D}(\text{One lane closure in units of a study period})] = \frac{34}{60\times6} = 0.0944$ study periods.

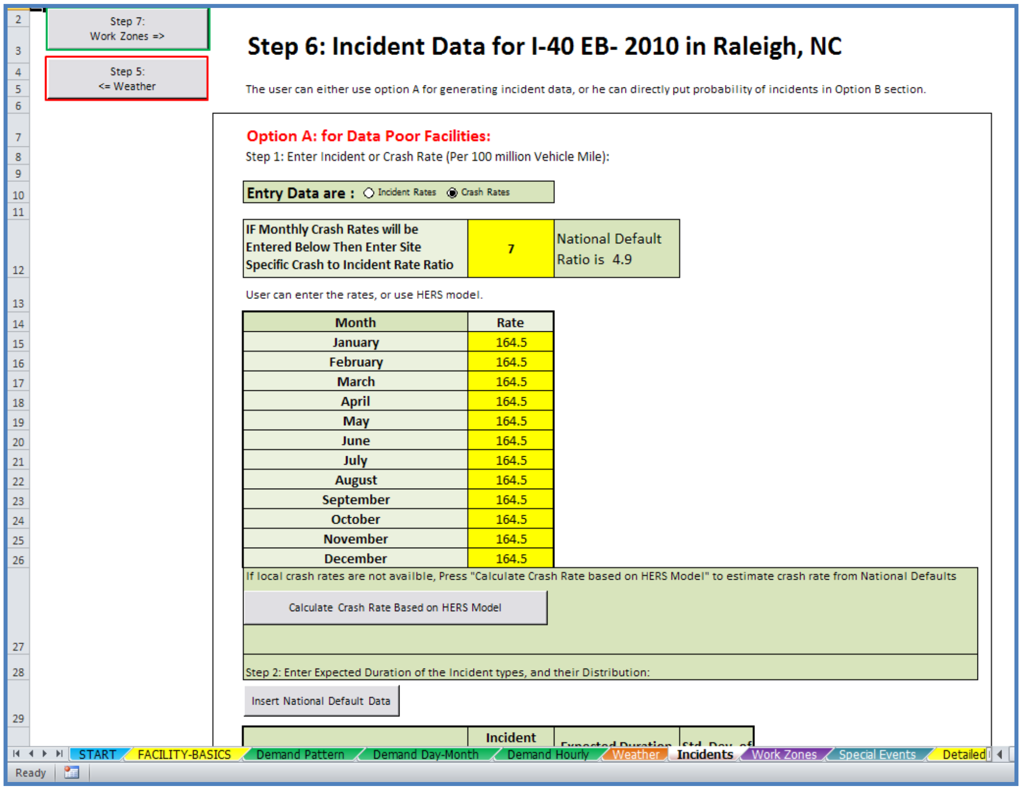
Alternatively, it can be shown that the expected number of incidents in month May (n_5) is the incident rate multiplied by the adjusted VMT, as shown by Equation F.19:

$$n_5 = \mathrm{IR}_5 \times (\mathrm{VMT} \times \mathrm{DM}_5) = 1{,}151.5 \times 330{,}006 \times 1.13$$

$$= 4.294 \text{ incidents/SP} \tag{F.19}$$

The factor $g(i)$ is also based on the use of national default values. Therefore, the portion of all incident times that incidents with one lane closure occur is 19.6%. The final probability of one lane closure in a study period in May can be

Table F.7. Schematic of FSG for I-40 EB Case Study Showing the Monthly Crash Rates and CTI



shown to be 7.64%. All computations are shown in Equation F.20:

$$\text{Prob}\{\text{single-lane closure incident in a study period in May}\}$$

$$= 1 - P_0 = 1 - e^{-(n_j \times g_i \times E[D(i)])}$$

$$= 1 - e^{-\left(4.294 \times 0.196 \times \frac{34}{60 \times 6}\right)} = 7.64\% \qquad \text{(F.20)}$$

This result is highlighted in Table F.9, where up to 60 such computations are made for each month and incident type combination. The probability balance is then assigned to the no-incident column.

An interesting sidelight of the application of Equation F.20 highlights the beneficial effects of good incident management practice. For example, by reducing the incident clearance time say, from 34 to 20 min, the probability of a lane closure incident decreases from 7.64% to 4.57%.

Table F.8. I-40 EB Demand Multipliers for Weekdays in May

	Number in May	Demand Multiplier
Monday	5	1.076
Tuesday	4	1.106
Wednesday	4	1.114
Thursday	4	1.158
Friday	4	1.210

Summary

This section documents a detailed methodology for estimating study period incident probabilities adjusted on a monthly basis. This approach ensures that the probabilities are unbiased and are founded on a definition that takes into account the number and expected duration of incidents, as well as the traffic demand. A key attribute of the method is its flexibility. It allows an agency to generate probability estimates consistent with their data availability, ranging from direct input of the probabilities from incident logs to estimation methods that use appropriate combinations of national and local defaults. Of course, a minimum amount of information about the facility geometry and AADTs is essential to applying the methodology.

Application of the method to a 12.5-mi freeway facility in North Carolina indicates that for a study period of 6 h in the p.m. peak encompassing all weekdays in 2010, there was approximately a 33% chance of an incident occurring. About two-thirds of the predicted incidents involved shoulder closures, and one-third involved lane closures. The impact of those incidents on travel time is documented elsewhere.

Table F.9. Final Study Period Monthly Incident Probabilities

Month	Probability of Different Incident Types					
	No Incident	Shoulder Closure	One-Lane Closure	Two-Lane Closure	Three-Lane Closure	Four-Lane Closure
January	66.42%	23.30%	7.06%	1.79%	1.43%	0.00%
February	66.36%	23.34%	7.08%	1.79%	1.43%	0.00%
March	65.10%	24.18%	7.36%	1.87%	1.49%	0.00%
April	63.79%	25.05%	7.66%	1.94%	1.56%	0.00%
May	63.87%	25.00%	**7.64%**	1.94%	1.55%	0.00%
June	64.53%	24.56%	7.49%	1.90%	1.52%	0.00%
July	64.10%	24.85%	7.59%	1.93%	1.54%	0.00%
August	65.30%	24.04%	7.32%	1.86%	1.48%	0.00%
September	65.97%	23.60%	7.17%	1.82%	1.45%	0.00%
October	65.04%	24.22%	7.38%	1.87%	1.50%	0.00%
November	66.79%	23.05%	6.98%	1.77%	1.41%	0.00%
December	68.56%	21.86%	6.59%	1.67%	1.33%	0.00%

The methodology could benefit from a number of enhancements, notably in acknowledging the correlation between incidents and weather conditions. The team is aware of a parallel effort under the auspices of SHRP 2 Project L04 in which a model is being tested in the New York area that provides conditional incident probabilities based on weather events. Another area of needed improvement is the correlation of (major) incidents and expected demand. These areas for improvement are recommended for future research and methodological development.

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APPENDIX G

Freeway Free-Flow Speed Adjustments for Weather, Incidents, and Work Zones

This appendix presents recommended free-flow speed adjustment factors (SAFs) for weather. The recommendations are based on a review of the literature and extraction of relevant data found in the literature.

HCM Definitions

This section presents the *2010 Highway Capacity Manual* (HCM2010) (Transportation Research Board of the National Academies 2010) definitions and values for freeway free-flow speed and capacity.

Free-Flow Speed

Chapter 10 of the HCM2010 defines free-flow speeds on freeways as "[t]he theoretical speed when the density and flow rate on the study segment are both zero. Chapter 11, Basic Freeway Segments, presents speed–flow curves that indicate that the free-flow speed on freeways is expected to prevail at flow rates between 0 and 1,000 passenger cars per hour per lane (pc/h/ln). In this broad range of flows, speed is insensitive to flow rates."

The free-flow speeds for dry pavement, fair weather, nonincident conditions define the base capacity for the freeway according to Exhibit 10-5 of the HCM2010. The relationship between free-flow speed and freeway base capacity is given in Table G.1.

The equivalent equation is given by Equation G.1:

$$\text{Base Capacity (pc/h/ln)} = 2{,}400 \text{ pc/h/ln} - 10 \times (70 - \min(70, \text{FFS})) \quad \text{(G.1)}$$

where FFS = free-flow speed under dry pavement, fair weather, nonincident conditions (mph).

Capacity and Speed at Capacity

Exhibit 11-2 in the HCM2010 defines capacity when traffic is at a density of 45 passenger cars per mile per lane for basic freeway segments under clear weather, dry pavement, nonincident conditions. The speed at capacity can then be derived from this information by using the basic speed–flow–density relationship. The speeds at capacity for different free-flow speeds are given in Table G.2.

The equivalent equation for the entries in this table is given by Equation G.2:

$$\text{Speed at Capacity (mph)} = [2{,}400 \text{ pc/h/ln} - 10 \times (70 - \min(70, \text{FFS}))]/45 \quad \text{(G.2)}$$

HCM Freeway Speed–Flow Curves

The clear weather, dry pavement speed–flow curves for basic freeway segments shown in Exhibit 11-2 of the HCM2010 can be approximated using the equations given in Exhibit 11-3 and shown here in Equation G.3:

$$S = \text{FFS if } v_p < BP\text{; otherwise,}$$

$$S = \text{FFS} - A \times (v_p - BP)^2 \quad \text{(G.3)}$$

where

S = speed at passenger car equivalent volume v_p (mph);
A = calibration parameter (see Table G.3);
B = breakpoint passenger car equivalent volume (pc/h/ln);
P = 1,000 + 200 × (75 − FFS) / 5; and
V = passenger car equivalent volume (pc/h/ln).

Equation G.3, however, does not provide for adjustments to the dry weather, nonincident capacity that can occur with bad weather or incidents. Equation 25-1 from Chapter 25 of the HCM2010 (shown here as Equation G.4) applies:

$$S = \text{FFS} + \left[1 - e^{\ln\left(\text{FFS} + 1 - \frac{C \times \text{CAF}}{45}\right) \times \frac{v_p}{C * \text{CAF}}}\right] \quad \text{(G.4)}$$

Table G.1. Relationship Between Free-Flow Speed and Freeway Base Capacity

Free-Flow Speed (mph)[a]	Base Capacity (pc/h/ln)
75	2,400
70	2,400
65	2,350
60	2,300
55	2,250

Source: HCM2010, Exhibit 10-5.
[a] Dry pavement, fair weather, nonincident.

Table G.3. HCM2010 Values for "A" Parameter in Freeway Free-Flow Speed Equations

FFS	A
75 mph	1.107×10^{-5}
70 mph	1.160×10^{-5}
65 mph	1.418×10^{-5}
60 mph	1.816×10^{-5}
55 mph	2.469×10^{-5}

Source: Exhibit 11-3, HCM2010.

where

S = segment speed (mph);
FFS = segment free-flow speed (mph);
C = original segment capacity (pc/h/ln);
CAF = capacity adjustment factor (unitless), subject to CAF > 0 and CAF < 45 × (FFS + 1)/C; and
v_p = segment flow rate (pc/h/ln).

Although HCM2010 Equation 25-1 is not precisely flat for passenger car equivalent volumes under 1,000 passenger cars per hour per lane (pcphpl), it is close enough for the purposes of speed and travel time prediction, and it has the advantage of being sensitive to capacity adjustments for weather, incidents, and work zones.

With a slight modification (the addition of a free-flow SAF to account for weather effects), HCM2010 Equation 25-1 can be used to predict speeds for weather, as well as incidents and work zones. This modification is shown in Equation G.5:

$$S = \text{FFS} \times \text{FAF} + \left[1 - e^{\ln\left(\text{FFS} \times \text{FAF} + 1 - \frac{C \times \text{CAF}}{45}\right) \times \frac{v_p}{C \times \text{CAF}}}\right] \qquad \text{(G.5)}$$

Table G.2. Dry Weather Speed at Capacity for Different Free-Flow Speeds

Free-Flow Speed (mph)	Capacity (pc/h/lane)[a]	Density at Capacity (pc/mi/ln)[b]	Speed at Capacity (mph)
75	2,400	45	53.3
70	2,400	45	53.3
65	2,350	45	52.2
60	2,300	45	51.1
55	2,250	45	50.0

Source: Computed from Exhibit 10-5, 2010 HCM.
[a] pc/h/lane = passenger cars per hour per lane.
[b] pc/mi/lane = passenger cars per mile per lane.

where all variables are the same as in Equation G.4, with the addition of FAF, the free-flow speed adjustment factor (unitless), which is subject to CAF > 0 and CAF < 45 × (FFS × FAF + 1) / C, and FAF > (C × CAF / 45 – 1) / FFS.

HCM Capacity Adjustments

The HCM capacity adjustments for weather, incidents, and work zones must be examined to ensure that the recommended free-flow SAFs do not fall below the limits set by Equation G.5.

Weather Capacity Adjustments

Exhibit 10-15 of the HCM2010 provides ranges and average capacity adjustments by weather type, based on research on Iowa freeways. This exhibit is shown as Table G.4. The implications for the minimum allowable free-flow SAF are shown in the right-hand columns of this table for freeways with dry weather free-flow speeds between 55 and 75 mph. Two extrapolations of the original HCM exhibit have been included here for weather conditions not explicitly covered in the original exhibit. Capacity adjustment factors (CAFs) for wet pavement, clear weather conditions have been set equivalent to light rain conditions. The capacity for light wind (<10 mph) conditions has been set equal to that for clear, dry pavement conditions (CAF = 1.00).

CAFs are applied to the base capacity as shown in Equation G.6:

$$\text{Base Capacity (Weather)} = \text{Base Capacity (Clear, Dry)} \times \text{CAF} \qquad \text{(G.6)}$$

where

Base Capacity (Weather) = Base capacity for inclement weather (pc/h/ln);

Table G.4. Weather Adjustments to Freeway Base Capacity

Weather Type		Capacity Adjustment Factors			Minimum Allowable Free-Flow Speed Adjustment Factors (According to Freeway Free-Flow Speed)				
		Low	High	Ave	55 mph	60 mph	65 mph	70 mph	75 mph
Clear	Dry Pavement	1.00	1.00	1.00	0.89	0.84	0.79	0.75	0.70
	Wet Pavement*	0.96	0.99	0.98	0.88	0.83	0.78	0.74	0.69
Rain	≤ 0.10 in/h	0.96	0.99	0.98	0.88	0.83	0.78	0.74	0.69
	≤ 0.25 in/h	0.90	0.94	0.93	0.84	0.78	0.74	0.70	0.66
	> 0.25 in/h	0.82	0.89	0.86	0.79	0.74	0.70	0.66	0.62
Snow	≤ 0.05 in/h	0.94	0.96	0.96	0.85	0.80	0.76	0.72	0.67
	≤ 0.10 in/h	0.88	0.94	0.91	0.84	0.78	0.74	0.70	0.66
	≤ 0.50 in/h	0.87	0.92	0.89	0.82	0.77	0.72	0.69	0.64
	> 0.50 in/h	0.72	0.79	0.78	0.70	0.66	0.62	0.59	0.55
Temp	< 50 deg F	0.99	0.99	0.99	0.88	0.83	0.78	0.74	0.69
	< 34 deg F	0.98	0.98	0.98	0.87	0.82	0.77	0.73	0.68
	< −4 deg F	0.90	0.93	0.91	0.83	0.78	0.73	0.69	0.65
Wind	< 10 mph*	1.00	1.00	1.00	0.89	0.84	0.79	0.75	0.70
	≤ 20 mph	0.99	0.99	0.99	0.88	0.83	0.78	0.74	0.69
	> 20 mph	0.98	0.99	0.98	0.88	0.83	0.78	0.74	0.69
Visibility	< 1 mi	N/A	N/A	0.93	0.83	0.78	0.73	0.69	0.65
	≤ 0.50 mi	N/A	N/A	0.88	0.78	0.73	0.69	0.66	0.61
	≤ 0.25 mi	N/A	N/A	0.89	0.79	0.74	0.70	0.66	0.62

Source: Exhibit 10-15, 2010 HCM (TRB 2010).
* Weather categories extrapolated as explained in text.
N/A = not applicable, data not available.

Base Capacity (Clear, Dry) = Base capacity for dry pavement, fair weather, non-incident conditions (pc/h/ln); and
CAF = capacity adjustment factor (unitless) (see Table G.5).

Incident Capacity Adjustments

Exhibit 10-17 of the HCM2010 provides recommended CAFs for incidents (see Table G.5).

The HCM CAFs are for the entire facility for differing numbers of lanes before and during the incident. These factors need to be translated into capacity per lane values for the lanes remaining open during the incident in order to be able to determine the appropriate minimum values for the free-flow SAFs. Table G.6 shows the CAFs in a capacity per open lane format after the conversion.

Table G.7 shows the minimum allowable free-flow SAFs for incidents on a freeway with a 55-mph free-flow speed and a base capacity of 2,250 pcphpl.

Work Zone Capacity Adjustments

Work zones include short-term work zone lane closures due to maintenance and long-term lane closures due to construction. According to the *Manual on Uniform Traffic Control Devices* (Federal Highway Administration 2009), construction duration for long-term work zones is more than 3 days and could last several weeks, months, or even years, depending on

Table G.5. Capacity Adjustment Factors According to "Before Incident" Conditions

Number of Lanes (One Direction)	Shoulder Disablement	Shoulder Accident	One Lane Blocked	Two Lanes Blocked	Three Lanes Blocked
2	0.95	0.81	0.35	0.00	N/A
3	0.99	0.83	0.49	0.17	0.00
4	0.99	0.85	0.58	0.25	0.13
5	0.99	0.87	0.65	0.40	0.20
6	0.99	0.89	0.71	0.50	0.26
7	0.99	0.91	0.75	0.57	0.36
8	0.99	0.93	0.78	0.63	0.41

Source: Exhibit 10-17, HCM2010.
N/A = not applicable, scenario not feasible.

Table G.6. Open Lane Capacity Adjustment Factors for Incidents

Lanes Open	Shoulder Disabled Vehicle	Shoulder Accident	One Lane Blocked	Two Lanes Blocked	Three Lanes Blocked
0	N/A	N/A	0.00	0.00	0.00
1	N/A	N/A	0.70	0.51	0.52
2	0.95	0.81	0.74	0.50	0.50
3	0.99	0.83	0.77	0.67	0.52
4	0.99	0.85	0.81	0.75	0.63
5	0.99	0.87	0.85	0.80	0.66
6	0.99	0.89	0.88	0.84	N/A

N/A = not applicable, data not available.

the nature of works. Short-term work zone duration is more than an hour and within a single daylight period (Federal Highway Administration 2009). Long-term construction zones generally use portable concrete barriers, while short-term work zones use standard channelizing devices.

Chapter 10 of the HCM2010 summarizes the lane closures and ranges of capacity during construction. Exhibit 10-14 of the HCM2010 provides work zone capacities in terms of vehicles per hour per lane according to the original number of lanes (before the work zone) and the number of lanes open when the work zone is in place.

In Table G.8, the passenger car per hour per lane equivalent is computed assuming level terrain, 5% heavy vehicles, and a 0.90 peak hour factor.

The vehicle per hour per lane capacities in Exhibit 10-14 of HCM2010 were converted to passenger car equivalents for the purpose of computing CAFs for work zones. CAFs for a freeway with a 65-mph free-flow speed were computed assuming that the values in Figure 10-14 of the HCM2010 apply to a freeway with a 65-mph free-flow speed and a base condition of dry weather and nonwork zone capacity of 2,300 pcphpl. The same CAFs computed for a freeway with a 65-mph

Table G.7. Minimum Free-Flow Speed Adjustment Factors for 55-mph Freeways

Lanes Open	Shoulder Disabled Vehicle	Shoulder Accident	One Lane Blocked	Two Lanes Blocked	Three Lanes Blocked
0	N/A	N/A	N/A	N/A	N/A
1	N/A	N/A	0.62	0.45	0.45
2	0.85	0.72	0.65	0.44	0.44
3	0.88	0.74	0.68	0.59	0.45
4	0.88	0.75	0.72	0.66	0.55
5	0.88	0.77	0.76	0.71	0.58
6	0.88	0.79	0.78	0.75	N/A

N/A = not applicable, data not available.

Table G.8. Capacities of Freeway Work Zones

Original Number of Lanes	One-Lane Work Zone	Two-Lane Work Zone	Three-Lane Work Zone
1 Lane Before	N/A	N/A	N/A
2 Lanes Before	1,400	N/A	N/A
3 Lanes Before	1,450	1,450	N/A
4 Lanes Before	1,350	1,450	1,500
Range	950–2,000	1,300–2,100	1,300–1,600
Average			
Vehicles per hour per lane	1,400	1,450	1,500
Passenger cars per hour per lane	1,590	1,650	1,710

Note: N/A = not applicable, data not available.
Source: Default values and ranges from Exhibit 10-14, HCM2010.

free-flow speed are assumed to apply to freeways with higher and lower free-flow speeds. In other words, the effect of the work zone on capacity is assumed to be proportional to the base capacity. The resulting CAFs applicable to all freeways, regardless of free-flow speed, are shown in the second column from the left in Table G.9. Exhibit 10-14 of the HCM2010 has been extrapolated to freeway work zones with five moving lanes. The right-hand five columns of Table G.9 show the equivalent minimum free-flow SAFs consistent with the computed CAFs.

Literature on Speed Effects

Weather Effects

During adverse weather—such as rain or snow—drivers usually slow down systemwide due to lower visibility and wet, icy, or slushy pavement conditions. Depending on the intensity of the rain or snow event, the speed adjustment can be little, noticeable, or significant. Researchers around the world have studied the effect of severe weather on free-flow speed. Their findings and average speed reductions calculated from the literature summary are presented in Table G.10. The low end of the reduction ranges could be applicable to light adverse weather or free-flow conditions, while the higher numbers could be applied to roadways with at-capacity volumes or under heavy rain or snow.

Strong et al. (2010) also conducted a thorough literature review on the topic. Among their most relevant findings is a study done by Japanese researchers, who found a 15% speed reduction for a blizzard condition, 18.6% for frozen pavement, 6.5% for snow flurries and snowfall, 6% for wet pavement, 11.3% for melting snow, 12% to 44% for compacted snow, and 15.4% for icy conditions.

Incident Effects

Data and literature on the effects of incidents on free-flow speeds are relatively rare and were not encountered in the limited literature research conducted for this appendix.

Work Zone Effects

The effects of work zones on free-flow speeds have not been examined in the literature. However, the effectiveness of work zone speed limits at reducing free-flow speeds within the work zone has been examined for different levels and methods of posting and enforcement.

Table G.9. Capacity and Minimum Free-Flow Speed Adjustment Factors for Work Zones

Work Zone Lanes Open	Work Zone CAF	Minimum Allowable Free-Flow Speed Adjustment Factors				
		55 mph (2,250 pc/h/ln)	60 mph (2,300 pc/h/ln)	65 mph (2,350 pc/h/ln)	70 mph (2,400 pc/h/ln)	75 mph (2,400 pc/h/ln)
1	0.68	0.60	0.56	0.53	0.50	0.47
2	0.70	0.62	0.58	0.55	0.52	0.49
3	0.72	0.64	0.60	0.57	0.54	0.50
4	0.74	0.66	0.62	0.58	0.55	0.52
5	0.77	0.68	0.64	0.60	0.57	0.53

Note: The minimum allowable free-flow speed adjustment factors are according to base free-flow speed and base capacity.
CAF = capacity adjustment factor.

Table G.10. Comparison and Summary of Literature Findings on Speed Reduction due to Weather

Researchers	Location	Weather Type			
		Rainfall	Wet Pavement	Snowfall	Icy Pavement
Kilpelainen and Summala (2007)	Finland	6–7 km/h[a]			
Koetse and Rietveld (2009)	N/A	N/A	Up to 25%	N/A	Up to 25%
Martin et al. (2000)	Utah (Arterials)	10%	13%	13%	30%
HraN/Aet al. (2006)	United States	3% [a,b] 9% [b,c]	N/A	5% [c]	N/A
Maze et al. (2006)	Minneapolis	6%	N/A	13%	N/A
Sabir et al. (2008)	the Netherlands	10–15% [d]	N/A	7%	N/A
Strong et al. (2010)	N/A	N/A	N/A	6 mph[c] 31 mph[f]	N/A
Rakha et al. (2008)	United States	3–6% [b,e] 8–10% [c,e] 6–9% [b,f] 8–14% [c,d]	N/A	5–16% [b,e] 5–16% [c,e] 5–19% [b,e]	N/A
Goodwin (2002)		10–25%	30–40%	10–25%	30–40%
Padget et al. (2001)	Iowa (Arterial)	N/A	N/A	N/A	18–20%
Average		**7–11%**	**19–21%**	**9–12%**	**22–24%**

Note: This table shows results for arterials as well as freeways.
[a] Under adverse weather and road conditions.
[b] Free-flow.
[c] At capacity.
[d] Rush.
[e] Light.
[f] Heavy.
N/A = not applicable, data not available.

This section summarizes past research efforts performed on work zone and speed and enforcement. Richards et al. (1985) studied several work zone speed control methods. Their study results indicate that flagging and law enforcement are effective methods for controlling speeds at work zones. The flagging treatment tested reduced speed an average of 19% for all sites, and the law enforcement treatment reduced speed an average of 18%.

Wasson et al. (2011) evaluated the temporal and spatial effects of work zone speed limit compliance over short 1- and 2-mi segments, as well as for the overall 12.2-mi work zone and approaching transition areas. Space mean speed was measured for approximately 11% of passing vehicles using 13 Bluetooth probe data acquisition stations. The presence of enforcement activities resulted in statistically significant reductions in the space mean speeds in the areas of enforcement and the adjacent highway segments. Although space mean speed was reduced by approximately 5 mph over the 12.2-mi segment during the enforcement activity, within 30 min of suspending the enforcement detail, the space mean speed increased and there was no statistically significant residual impact on the space mean speed for the 12.2-mi segment.

Hou et al. (2011) conducted field studies on three I-70 maintenance short-term work zones in rural Missouri for three speed limit scenarios: no posted speed limit reduction, a 10-mph posted speed limit reduction, and a 20-mph posted speed limit reduction. The observed 85th percentile speeds were 81, 62, and 48 mph for no posted speed limit reduction, a 10-mph posted speed limit reduction, and a 20-mph posted speed limit reduction, respectively. The percentage of drivers who exceeded the posted speed limit by over 10 mph were 15.4%, 4.8%, and 0.9% with no speed limit reduction, a 10-mph posted speed limit reduction, and a 20-mph posted speed limit reduction, respectively. Researchers concluded that a reduction in posted speed limit was effective in reducing prevailing speeds in Missouri.

Brewer et al. (2005) tested three devices: speed display trailer, changeable message sign with radar, and orange-border speed limit sign. They found that devices that display an approaching driver's speed are effective at reducing speed and improving work zone speed compliance. In the absence of active work taking place and when the road maintains a normal cross section, drivers generally maintain the speed they were traveling before entering the work zone, regardless of the posted work zone speed limit. Officials should post the realistic speed limit to avoid work zone speed limits that drivers ignore or widely disobey, and the speed limits should be confined as much as possible to the specific areas where active work is taking place.

Franz and Chang (2011) evaluated the effectiveness of an automated speed enforcement system in work zones. Before versus during enforcement periods analysis showed a general reduction in speeding by aggressive motorists, while creating a more stable speeding distribution through the work zone. The comparison of during versus after enforcement periods showed that motorists may learn where enforcement is taking place and adjust their speed accordingly.

Li et al. (2010) evaluated the effectiveness of a portable changeable message sign (PCMS) in reducing vehicle speeds in the upstream of one-lane, two-way work zones on rural highways. The evaluation was performed under three conditions during field experiments: PCMS switched on, PCMS switched off but still visible, and PCMS removed from the road and out of sight. The results indicated that the PCMS, whether turned on or off, was significantly more effective than the PCMS absent from the highway. Vehicle speeds were reduced by 4.7 mph and 3.3 mph when the PCMS was turned on and off, respectively. When the PCMS was absent from the road, the speed reduction was 1.9 mph.

Hajbabaie et al. (2009) compared the effects of four speed management techniques: speed feedback trailer, police car, speed feedback trailer plus police car, and automated speed photo-radar enforcement. All the law enforcement methods significantly reduced the mean speed of free-flowing cars by 6.1 to 8.4 mph in the median lane and by 4.2 to 6.9 mph in the shoulder lane. In the moderately speeding site, police and speed photo-radar enforcement reduced the mean speeds similarly in both lanes; however, trailer plus police car treatment resulted in even larger speed reductions.

Theiss et al. (2010) conducted a study on the operational effectiveness of electronic speed limit signs and flexible roll-up work zone speed limit signs. Researchers concluded from the long-term field study that motorists understood and appreciated the intent of the electronic speed limit signs. The short-term field study of both the electronic speed limit and flexible roll-up work zone speed limit signs resulted in lower mean speeds and percentage of vehicles exceeding the speed limit downstream of the reduced work zone speed limit compared with standard temporary speed limit signing. The researchers recommended the use of electronic and flexible roll-up work zone speed limit signs to better manage short-term speed limits because of the simplicity this signage offers in varying speed limits to match conditions.

Recommended Free-Flow Speed Adjustments

This section presents the recommended freeway free-flow speed adjustments for the effects of weather, incidents, and work zones.

Weather Free-Flow Speed Adjustments

Based on the preceding information, the free-flow speed reductions shown in Table G.11 are recommended for adverse weather conditions on urban and rural freeways. The weather categories in Table G.11 are adapted from Exhibit 10-15 of the HCM2010. The free-flow speed at base condition is under clear weather, dry pavement, and nonincident conditions. All

Table G.11. Recommended Freeway Free-Flow Speed Adjustment Factors for Weather

Weather Type		Clear Weather, Dry Pavement Free-Flow Speeds				
		55 mph	60 mph	65 mph	70 mph	75 mph
Clear	Dry Pavement	1.00	1.00	1.00	1.00	1.00
	Wet Pavement	0.97	0.96	0.96	0.95	0.94
Rain	≤ 0.10 in/h	0.97	0.96	0.96	0.95	0.94
	≤ 0.25 in/h	0.96	0.95	0.94	0.93	0.93
	> 0.25 in/h	0.94	0.93	0.93	0.92	0.91
Snow	≤ 0.05 in/h	0.94	0.92	0.89	0.87	0.84
	≤ 0.10 in/h	0.92	0.90	0.88	0.86	0.83
	≤ 0.50 in/h	0.90	0.88	0.86	0.84	0.82
	> 0.50 in/h	0.88	0.86	0.85	0.83	0.81
Temp	< 50 deg F	0.99	0.99	0.99	0.98	0.98
	< 34 deg F	0.99	0.98	0.98	0.98	0.97
	< -4 deg F	0.95	0.95	0.94	0.93	0.92
Wind	< 10 mph	1.00	1.00	1.00	1.00	1.00
	≤ 20 mph	0.99	0.98	0.98	0.97	0.96
	> 20 mph	0.98	0.98	0.97	0.97	0.96
Visibility	< 1 mi	0.96	0.95	0.94	0.94	0.93
	≤ 0.50 mi	0.95	0.94	0.93	0.92	0.91
	≤ 0.25 mi	0.95	0.94	0.93	0.92	0.91

Table G.12. Enforcement Adjustment Factors for Work Zone Free-Flow Speeds

Enforcement Measure	Enforcement Adjustment Factor (%)
Static signs	50
Flagmen	70
Dynamic feedback signs	80
Visibly present enforcement personnel	90
Feedback signs plus visibly present enforcement personnel	100

the recommended free-flow SAFs equal or exceed the minimum values given in Exhibit 10-15 of the HCM2010.

Rakha et al. (2008), who produced one of the few papers to isolate free-flow speed effects from capacity speed effects, were the primary source for the free-flow speed adjustments in Table G.11. The higher end of the range of percentage adjustments was assumed to apply to the freeways with the highest free-flow speeds under clear weather, dry pavement conditions. Their heavy rain and heavy snowfall adjustments were assumed to apply to the highest levels of rainfall and snowfall in the Iowa study cited in Exhibit 10-15 of the HCM2010. Their light rain and light snow adjustments were assumed to apply to the lowest rainfall and snowfall categories in Exhibit 10-15. Speed adjustment values for intermediate rainfall and snowfall rates were interpolated between their high and low values.

The free-flow speed for any weather categories can be derived by multiplying the clear weather, dry pavement free-flow speed for the facility by the free-flow SAF for the appropriate weather event in Table G.11.

Incident Free-Flow Speed Adjustments

Due to the lack of data on free-flow speeds in incident zones, it is recommended that a nominal free-flow SAF of 1.00 be used. It may be reduced at the discretion of the analyst to reflect the possible effects of rubbernecking.

Work Zone Free-Flow Speed Adjustments

The effects on free-flow speeds of narrower lanes and reduced right-side lateral clearances within the work zone (see Chapter 11 of the HCM2010) are presumed to be accounted for in the selected reduced posted speed limit for the work zone.

The work zone free-flow speed is then the before work free-flow speed adjusted for changes in the posted speed limit through the work zone. The effectiveness of the work zone speed limit at reducing free-flow speed is discounted according to the degree of visibility of the speed limits and the degree of enforcement within the work zone. The computation is shown by Equation G.7:

$$FFS_{WZ} = FFS_{HCM} + [PSL_{WZ} - PSL_{NWZ}] \times F_{ENF} \quad \text{(G.7)}$$

where

FFS_{WZ} = Free-flow speed within the work zone;
FFS_{HCM} = Geometrically determined free-flow speed computed or field-measured per HCM;
PSL_{WZ} = Posted speed limit within the work zone;
PSL_{NWZ} = Posted speed limit without the work zone; and
F_{ENF} = Enforcement adjustment factor to account for the effects of different levels of signing and enforcement of the work zone speed limit. Based on the literature, the enforcement adjustment factors shown in Table G.12 are recommended.

The free-flow SAF for the work zone is then the estimated work zone free-flow speed divided by the before work zone free-flow speed.

If there is no change in the posted speed limit for the facility within the work zone, then there is no change in the free-flow speed within the work zone. The free-flow SAF in this case is 1.00.

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APPENDIX H

Default Factors for the Urban Streets Reliability Methodology

This appendix documents the default values used in the various procedures of the scenario generation stage of the urban streets reliability methodology.

Weather Event Procedure

The weather event procedure is based on the weather characteristics identified in the following list. Default values are provided for these characteristics in the software implementation of the reliability methodology (National Climatic Data Center 2011a; 2011b; 2011c).

- Total normal precipitation;
- Total normal snowfall;
- Number of days with precipitation of 0.01 in. or more; and
- Precipitation rate.

The default data for 284 U.S. cities and territories are provided. Table H.1 illustrates the mean number of days with precipitation for 35 of these cities. The values shown represent an average for several years at each city (i.e., minimum of 17 years, maximum of 125 years, average 61 years).

Table H.2 illustrates the default total snowfall for 35 cities. The values shown represent an average for several years at each city (i.e., minimum of 11 years, maximum of 142 years, average 59 years).

Table H.3 illustrates the default normal daily mean temperature for 35 cities. The values shown represent an average for 30 years at each city. Table H.4 illustrates the default normal precipitation data for the same 35 cities. The values shown represent 30-year averages.

Table H.5 illustrates the default average precipitation rate for 35 cities. The values shown represent an average for 5 to 10 years at each city.

Traffic Demand Variation Procedure

This section lists the default values for the hour-of-day, day-of-week, and month-of-year factors provided in the reliability methodology. They are based on research by Hallenbeck et al. (1997). They were found to vary by roadway functional class and by vehicle class. The functional classes considered are identified in the following list. The number associated with each class corresponds to the column headings in Tables H.6 and H.8.

- Rural interstate (1);
- Rural principal arterial (2);
- Rural minor arterial (6);
- Rural major collector (7);
- Rural minor collector (8);
- Urban interstate (11);
- Urban other freeway and expressway (12);
- Urban principal arterial (14); and
- Urban minor arterial (16).

The hour-of-day factors are multiplied by an annual average daily traffic (AADT) volume to estimate the annual average hourly volume. The factors for passenger cars are listed in Table H.6. This vehicle class was found to represent 65% to 75% of the traffic stream. The hour-of-day factors for the other vehicle classes show a similar variation. These factors were obtained from the tables in Hallenbeck et al. (1997, pp. 69–82).

The day-of-week factors are multiplied by the AADT volume to estimate the annual average daily volume for a given day of week. The factors for passenger cars are listed in Table H.7. These factors were obtained from Table 3 of Hallenbeck et al. (1997).

The month-of-year factors are multiplied by the AADT volume to estimate the annual average daily volume for a given month. The factors for four vehicle classes combined

(text continues on page 229)

Table H.1. Default Mean Number of Days with Precipitation

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
BIRMINGHAM AP, AL	11	10	10	9	9	10	12	9	7	6	8	10
HUNTSVILLE, AL	11	9	11	9	10	9	10	8	8	7	9	10
MOBILE, AL	10	9	9	7	8	11	15	13	9	6	7	9
MONTGOMERY, AL	10	9	9	7	8	9	11	9	7	5	7	9
FLAGSTAFF, AZ	7	7	8	5	4	2	11	12	6	5	5	6
PHOENIX, AZ	3	4	3	1	0	0	4	4	2	2	2	3
TUCSON, AZ	4	3	3	1	1	1	9	9	4	3	2	4
WINSLOW, AZ	4	4	4	3	2	2	6	8	5	3	3	4
YUMA, AZ	3	2	2	1	*	*	1	2	1	1	1	2
FORT SMITH, AR	7	7	9	9	10	8	7	6	7	7	7	7
LITTLE ROCK, AR	9	8	10	9	10	8	8	6	7	7	8	9
NORTH LITTLE ROCK, AR	9	9	9	9	11	8	8	6	7	7	8	9
BAKERSFIELD, CA	6	6	6	4	1	0	0	0	1	2	4	5
BISHOP, CA	3	3	2	2	2	1	1	1	1	1	2	2
EUREKA, CA	16	14	15	12	8	5	2	2	4	8	13	15
FRESNO, CA	7	7	6	4	2	0	1	1	1	2	5	7
LONG BEACH, CA	5	5	5	3	1	1	1	1	1	2	3	5
LOS ANGELES AP, CA	6	6	5	3	1	0	0	1	1	2	3	5
LOS ANGELES C.O., CA	6	5	5	3	1	0	1	1	1	2	3	5
MOUNT SHASTA, CA	12	11	12	9	7	5	2	2	3	6	10	12
REDDING, CA	13	11	10	8	6	3	0	0	1	4	8	12
SACRAMENTO, CA	10	9	8	5	3	1	1	1	1	3	7	9
SAN DIEGO, CA	6	6	6	4	1	0	1	0	1	2	4	6
SAN FRANCISCO AP, CA	11	10	9	5	2	1	1	1	1	3	7	10
SAN FRANCISCO C.O., CA	11	10	10	6	3	1	1	1	2	4	8	10
SANTA BARBARA, CA	5	6	6	2	1	0	0	1	1	2	3	5
SANTA MARIA, CA	7	7	7	4	1	0	1	1	1	2	5	7
STOCKTON, CA	9	8	8	5	2	0	1	1	1	3	7	7
ALAMOSA, CO	3	3	5	5	5	5	8	10	6	4	3	4
COLORADO SPRINGS, CO	4	4	7	7	10	9	12	12	6	4	4	4
DENVER, CO	5	5	8	8	10	8	9	8	6	5	5	5
GRAND JUNCTION, CO	6	6	7	6	6	3	4	6	6	5	5	5
PUEBLO, CO	4	4	6	6	8	7	9	8	4	3	3	3

*Missing data.

Table H.2. Default Total Snowfall (inches)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
BIRMINGHAM AP, AL	0.6	0.2	0.3	0.1	T	T	T	0	T	T	T	0.3
HUNTSVILLE, AL	1.4	0.8	0.4	T	T	T	0	T	0	T	0	0.2
MOBILE, AL	0.1	0.1	0.1	T	T	0	T	0	0	0	T	0.1
MONTGOMERY, AL	0.2	0.1	0.1	0	T	0	0	0	0	T	T	0
FLAGSTAFF, AZ	21.2	19.2	20.8	9	1.8	T	T	T	0.1	2.2	9.6	17
PHOENIX, AZ	T	0	T	T	T	0	0	0	0	T	0	T
TUCSON, AZ	0.3	0.2	0.2	0.1	T	0	T	T	T	T	0.1	0.3
WINSLOW, AZ	2.5	1.8	1.9	0.4	0	0	0	0	T	0.2	0.7	3
YUMA, AZ	0	0	0	0	0	0	0	0	0	0	0	T
FORT SMITH, AR	2.5	1.8	0.7	T	T	T	0	0	0	T	0.4	0.8
LITTLE ROCK, AR	2.4	1.6	0.5	T	T	T	0	0	0	T	0.2	0.6
NORTH LITTLE ROCK, AR	2.5	2.4	0.6	T	T	T	T	0	0	T	0.3	0.5
BAKERSFIELD, CA	T	T	0	T	0	0	0	0	0	0	0	T
BISHOP, CA	4	1.5	0.7	0.3	0.1	0	0	0	T	0	0.3	1.3
EUREKA, CA	0.1	0.1	T	T	0	0	0	0	0	0	0	T
FRESNO, CA	0.1	T	T	0	0	T	0	0	0	T	0	0
LONG BEACH, CA	T	T	0	0	0	0	0	0	0	0	0	0
LOS ANGELES AP, CA	T	T	T	0	0	0	0	0	0	0	0	T
LOS ANGELES C.O., CA	0	T	0	0	0	0	0	0	0	0	0	T
MOUNT SHASTA, CA	29.9	16.9	17.1	8.9	0.8	T	0	0	0	0.4	9	21.9
REDDING, CA	1.5	0.3	0.2	T	0.2	T	0	T	0	0	T	2
SACRAMENTO, CA	T	T	T	0	T	0	0	0	0	0	0	T
SAN DIEGO, CA	T	0	T	T	0	0	0	0	0	0	T	T
SAN FRANCISCO AP, CA	0	T	T	0	0	0	0	0	0	0	0	0
SAN FRANCISCO C.O., CA	T	T	T	0	0	0	0	0	0	0	0	T
SANTA BARBARA, CA	0	0	0	0	0	0	0	0	0	0	0	0
SANTA MARIA, CA	T	T	T	0	0	0	0	0	0	0	T	T
STOCKTON, CA	0	0	T	T	T	0	0	0	0	0	0	0
ALAMOSA, CO	4.5	4.2	5.5	4.4	1.8	0	T	0	0.2	3	4	5.2
COLORADO SPRINGS, CO	5.2	4.7	8.9	6.3	1.5	T	T	T	1	3.4	4.9	5.2
DENVER, CO	7.9	7.4	12.2	8.5	1.6	0	T	T	1.6	4	8.7	7.8
GRAND JUNCTION, CO	6.6	3.8	3.4	1.2	0.1	T	T	T	0.1	0.5	2.8	5.1
PUEBLO, CO	5.8	4.2	6.7	3.7	0.6	T	T	T	0.6	1.4	4.1	5.3

Note: T = trace.

Table H.3. Default Normal Daily Mean Temperature (°F)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
BIRMINGHAM AP, AL	42.6	46.8	54.5	61.3	69.3	76.4	80.2	79.6	73.8	62.9	53.1	45.6
HUNTSVILLE, AL	39.8	44.3	52.3	60.4	68.6	76	79.5	78.6	72.4	61.3	51.2	43.1
MOBILE, AL	50.1	53.5	60.2	66.1	73.5	79.3	81.5	81.3	77.2	67.7	58.9	52.3
MONTGOMERY, AL	46.6	50.5	57.9	64.3	72.3	78.9	81.8	81.2	76.3	65.4	56.1	49
FLAGSTAFF, AZ	29.7	32.2	36.6	42.9	50.8	60.1	66.1	64.4	57.8	47.1	36.5	30.2
PHOENIX, AZ	56.1	59.9	64.6	71.2	80.7	89.8	94.8	93.1	87.3	74.9	62.7	55.5
TUCSON, AZ	51.7	55	59.2	66	74.5	84.1	86.5	84.9	80.9	70.5	58.7	51.9
WINSLOW, AZ	34.2	40	46.3	53.4	62.2	72.1	77.5	75.6	68.2	55.9	43.2	34.1
YUMA, AZ	58.1	62	66.5	72.7	79.9	88.8	94.1	93.5	88.2	77.2	64.8	57.4
FORT SMITH, AR	38	43.7	52.6	61.1	69.5	77.5	82.2	81.5	73.9	62.8	50.5	41
LITTLE ROCK, AR	40.1	45.2	53.4	61.4	70.1	78.4	82.4	81.3	74.4	63.3	51.7	43.2
NORTH LITTLE ROCK, AR	40.2	45.6	54.3	63	70.9	78.8	83.2	82.1	75	64.5	52.5	43.4
BAKERSFIELD, CA	47.8	53.3	57.3	62.7	70.3	77.7	83.1	81.9	76.7	67.2	54.8	47.2
BISHOP, CA	38	42.4	47.7	54.1	62.5	71.1	76.8	74.8	67.3	56.6	44.8	38
EUREKA, CA	47.9	48.9	49.2	50.7	53.6	56.3	58.1	58.7	57.4	54.5	51	47.9
FRESNO, CA	46	51.4	55.5	61.2	68.8	76.1	81.4	79.9	74.6	65	52.7	45.2
LONG BEACH, CA	57	58.3	59.7	63	65.9	69.8	73.8	75.1	73.4	68.6	61.8	57.1
LOS ANGELES AP, CA	57.1	58	58.3	60.8	63.1	66.4	69.3	70.7	70.1	66.9	61.6	57.6
LOS ANGELES C.O., CA	58.3	60	60.7	63.8	66.2	70.5	74.2	75.2	74	69.5	62.9	58.5
MOUNT SHASTA, CA	35.3	38.2	41.2	46.3	53.2	60.2	66.1	65.1	59.5	50.5	39.9	34.8
REDDING, CA	45.5	49.1	52.5	57.8	66.2	75.2	81.3	78.9	73.4	63.2	51.1	45.3
SACRAMENTO, CA	46.3	51.2	54.5	58.9	65.5	71.5	75.4	74.8	71.7	64.4	53.3	45.8
SAN DIEGO, CA	57.8	58.9	60	62.6	64.6	67.4	70.9	72.5	71.6	67.6	61.8	57.6
SAN FRANCISCO AP, CA	49.4	52.4	54	56.2	58.7	61.4	62.8	63.6	63.9	61	54.7	49.5
SAN FRANCISCO C.O., CA	52.3	55	55.9	57.3	58.4	60.5	61.3	62.4	63.7	62.5	57.5	52.7
SANTA BARBARA, CA	53.1	55.2	56.7	58.9	60.9	64.2	67	68.6	67.4	63.5	57.5	53.2
SANTA MARIA, CA	51.6	53.1	53.8	55.5	57.8	60.9	63.5	64.2	63.9	61.1	55.5	51.6
STOCKTON, CA	46	51.1	54.9	60	66.7	73.2	77.3	76.5	72.8	64.6	53.1	45.3
ALAMOSA, CO	14.7	22.5	32.7	40.8	50.4	59.4	64.1	62.1	54.5	42.8	28.4	17.1
COLORADO SPRINGS, CO	28.1	31.7	37.8	45.3	54.6	64.4	69.6	67.6	59.8	48.9	36.2	29
DENVER, CO	29.2	33.2	39.6	47.6	57.2	67.6	73.4	71.7	62.4	51	37.5	30.3
GRAND JUNCTION, CO	26.1	34.1	43.4	50.9	60.5	71.1	76.8	74.7	65.4	52.7	38.1	28.2
PUEBLO, CO	29.3	34.6	41.8	49.9	59.7	69.8	75.4	73.5	64.8	52.4	38.4	30.3

Table H.4. Default Normal Precipitation (inches)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
BIRMINGHAM AP, AL	5.45	4.21	6.1	4.67	4.83	3.78	5.09	3.48	4.05	3.23	4.63	4.47
HUNTSVILLE, AL	5.52	4.95	6.68	4.54	5.24	4.22	4.4	3.32	4.29	3.54	5.22	5.59
MOBILE, AL	5.75	5.1	7.2	5.06	6.1	5.01	6.54	6.2	6.01	3.25	5.41	4.66
MONTGOMERY, AL	5.04	5.45	6.39	4.38	4.14	4.13	5.31	3.63	4.22	2.58	4.53	4.97
FLAGSTAFF, AZ	2.18	2.56	2.62	1.29	0.8	0.43	2.4	2.89	2.12	1.93	1.86	1.83
PHOENIX, AZ	0.83	0.77	1.07	0.25	0.16	0.09	0.99	0.94	0.75	0.79	0.73	0.92
TUCSON, AZ	0.99	0.88	0.81	0.28	0.24	0.24	2.07	2.3	1.45	1.21	0.67	1.03
WINSLOW, AZ	0.46	0.53	0.61	0.27	0.36	0.3	1.18	1.31	1.02	0.9	0.55	0.54
YUMA, AZ	0.38	0.28	0.27	0.09	0.05	0.02	0.23	0.61	0.26	0.26	0.14	0.42
FORT SMITH, AR	2.37	2.59	3.94	3.91	5.29	4.28	3.19	2.56	3.61	3.94	4.8	3.39
LITTLE ROCK, AR	3.61	3.33	4.88	5.47	5.05	3.95	3.31	2.93	3.71	4.25	5.73	4.71
NORTH LITTLE ROCK, AR	3.37	3.27	4.88	5.03	5.4	3.51	3.15	2.97	3.53	3.81	5.74	4.53
BAKERSFIELD, CA	1.18	1.21	1.41	0.45	0.24	0.12	0	0.08	0.15	0.3	0.59	0.76
BISHOP, CA	0.88	0.97	0.62	0.24	0.26	0.21	0.17	0.13	0.28	0.2	0.44	0.62
EUREKA, CA	5.97	5.51	5.55	2.91	1.62	0.65	0.16	0.38	0.86	2.36	5.78	6.35
FRESNO, CA	2.16	2.12	2.2	0.76	0.39	0.23	0.01	0.01	0.26	0.65	1.1	1.34
LONG BEACH, CA	2.95	3.01	2.43	0.6	0.23	0.08	0.02	0.1	0.24	0.4	1.12	1.76
LOS ANGELES AP, CA	2.98	3.11	2.4	0.63	0.24	0.08	0.03	0.14	0.26	0.36	1.13	1.79
LOS ANGELES C.O., CA	3.33	3.68	3.14	0.83	0.31	0.06	0.01	0.13	0.32	0.37	1.05	1.91
MOUNT SHASTA, CA	7.06	6.45	5.81	2.65	1.87	0.99	0.39	0.43	0.87	2.21	5.08	5.35
REDDING, CA	6.5	5.49	5.15	2.4	1.66	0.69	0.05	0.22	0.48	2.18	4.03	4.67
SACRAMENTO, CA	3.84	3.54	2.8	1.02	0.53	0.2	0.05	0.06	0.36	0.89	2.19	2.45
SAN DIEGO, CA	2.28	2.04	2.26	0.75	0.2	0.09	0.03	0.09	0.21	0.44	1.07	1.31
SAN FRANCISCO AP, CA	4.45	4.01	3.26	1.18	0.38	0.11	0.03	0.07	0.2	1.04	2.49	2.89
SAN FRANCISCO C.O., CA	4.72	4.15	3.4	1.25	0.54	0.13	0.04	0.09	0.28	1.19	3.31	3.18
SANTA BARBARA, CA	3.57	4.28	3.51	0.63	0.23	0.05	0.03	0.11	0.42	0.52	1.32	2.26
SANTA MARIA, CA	2.64	3.23	2.94	0.91	0.32	0.05	0.03	0.05	0.31	0.45	1.24	1.84
STOCKTON, CA	2.71	2.46	2.28	0.96	0.5	0.09	0.05	0.05	0.33	0.82	1.77	1.82
ALAMOSA, CO	0.25	0.21	0.46	0.54	0.7	0.59	0.94	1.19	0.89	0.67	0.48	0.33
COLORADO SPRINGS, CO	0.28	0.35	1.06	1.62	2.39	2.34	2.85	3.48	1.23	0.86	0.52	0.42
DENVER, CO	0.51	0.49	1.28	1.93	2.32	1.56	2.16	1.82	1.14	0.99	0.98	0.63
GRAND JUNCTION, CO	0.6	0.5	1	0.86	0.98	0.41	0.66	0.84	0.91	1	0.71	0.52
PUEBLO, CO	0.33	0.26	0.97	1.25	1.49	1.33	2.04	2.27	0.84	0.64	0.58	0.39

Table H.5. Default Average Precipitation Rate (inches/hour)

LOCATION	YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
BIRMINGHAM AP, AL	1999	0.099	0.1	0.116	0.113	0.165	0.151	0.161	0.185	0.151	0.135	0.105	0.096
HUNTSVILLE, AL	1999	0.071	0.077	0.094	0.092	0.157	0.143	0.196	0.153	0.134	0.092	0.08	0.087
MOBILE, AL	1999	0.116	0.098	0.133	0.148	0.108	0.185	0.167	0.199	0.148	0.112	0.154	0.129
MONTGOMERY, AL	1999	0.105	0.117	0.141	0.132	0.158	0.171	0.211	0.17	0.143	0.12	0.091	0.085
FLAGSTAFF, AZ	1999	0.028	0.038	0.04	0.047	0.038	0.056	0.078	0.091	0.083	0.052	0.045	0.038
PHOENIX, AZ	1999	0.053	0.043	0.044	0.024	0.06	0.11	0.16	0.13	0.121	0.095	0.086	0.045
TUCSON, AZ	1999	0.036	0.044	0.033	0.034	0.057	0.127	0.087	0.158	0.165	0.074	0.045	0.041
WINSLOW, AZ	1999	0.033	0.029	0.04	0.065	0.05	0.133	0.144	0.08	0.058	0.053	0.049	0.019
YUMA, AZ	1996	0.048	0.069	0.037	0.033	0.067	0.025	0.394	0.159	0.12	0.07	0.057	0.039
FORT SMITH, AR	1999	0.054	0.079	0.081	0.115	0.107	0.18	0.172	0.131	0.096	0.099	0.116	0.087
LITTLE ROCK, AR	1999	0.114	0.115	0.111	0.143	0.132	0.178	0.158	0.264	0.115	0.218	0.152	0.12
NORTH LITTLE ROCK, AR	1999	0.074	0.069	0.1	0.102	0.113	0.147	0.211	0.184	0.082	0.1	0.091	0.072
BAKERSFIELD, CA	1999	0.046	0.042	0.051	0.04	0.033	0.075	0.048	0.048	0.021	0.059	0.048	0.032
BISHOP, CA	1999	0.06	0.045	0.034	0.038	0.044	0.048	0.075	0.125	0.014	0.052	0.032	0.046
EUREKA, CA	1999	0.043	0.038	0.043	0.036	0.038	0.035	0.025	0.036	0.062	0.046	0.048	0.047
FRESNO, CA	1999	0.043	0.044	0.041	0.037	0.11	0.138	0.22	0.156	0.092	0.049	0.061	0.046
LONG BEACH, CA	1999	0.054	0.057	0.059	0.068	0.073	0.075	0.014	0.0325	0.051	0.046	0.051	0.076
LOS ANGELES AP, CA	1999	0.054	0.06	0.067	0.059	0.068	0.087	0.029	0.0305	0.032	0.09	0.086	0.08
LOS ANGELES C.O., CA	1999	0.064	0.074	0.066	0.072	0.05	0.075	0.022	0.0365	0.051	0.038	0.06	0.076
MOUNT SHASTA, CA	1999	0.076	0.08	0.084	0.085	0.078	0.114	0.122	0.115	0.116	0.083	0.082	0.082
REDDING, CA	1999	0.078	0.107	0.107	0.08	0.079	0.112	0.072	0.14	0.19	0.09	0.067	0.079
SACRAMENTO, CA	1999	0.045	0.055	0.043	0.04	0.04	0.081	0.064	0.047	0.042	0.073	0.05	0.051
SAN DIEGO, CA	1999	0.052	0.052	0.055	0.043	0.032	0.048	0.119	0.0895	0.06	0.07	0.056	0.058
SAN FRANCISCO AP, CA	1999	0.048	0.049	0.032	0.038	0.041	0.022	0.041	0.06	0.03	0.083	0.037	0.043
SAN FRANCISCO C.O., CA	1999	0.045	0.045	0.036	0.043	0.04	0.026	0.0365	0.047	0.059	0.098	0.049	0.051
SANTA BARBARA, CA	1999	0.079	0.12	0.113	0.097	0.07	0.1	0.067	0.1	0.1	0.124	0.115	0.087
SANTA MARIA, CA	1999	0.041	0.048	0.063	0.036	0.031	0.076	0.023	0.0465	0.07	0.057	0.062	0.044
STOCKTON, CA	1999	0.047	0.049	0.041	0.031	0.063	0.029	0.053	0.053	0.077	0.05	0.061	0.031
ALAMOSA, CO	1999	0.017	0.034	0.026	0.034	0.076	0.058	0.094	0.084	0.045	0.034	0.027	0.035
COLORADO SPRINGS, CO	1999	0.029	0.038	0.032	0.044	0.053	0.109	0.126	0.103	0.058	0.037	0.037	0.046
DENVER, CO	1999	0.044	0.041	0.058	0.057	0.063	0.115	0.192	0.125	0.076	0.072	0.052	0.058
GRAND JUNCTION, CO	1999	0.025	0.022	0.031	0.037	0.041	0.042	0.088	0.061	0.054	0.037	0.033	0.023
PUEBLO, CO	1999	0.02	0.023	0.037	0.05	0.078	0.098	0.103	0.117	0.147	0.033	0.033	0.038

Table H.6. Default Hour-of-Day Factors for Passenger Cars

		Functional Class								
Category	**Hour**	**1**	**2**	**6**	**7**	**8**	**11**	**12**	**14**	**16**
Weekday	0	0.011	0.009	0.012	0.012	0.008	0.011	0.010	0.010	0.010
	1	0.008	0.005	0.006	0.005	0.004	0.006	0.006	0.006	0.006
	2	0.006	0.003	0.003	0.004	0.001	0.005	0.004	0.005	0.004
	3	0.006	0.003	0.003	0.003	0.001	0.004	0.004	0.005	0.002
	4	0.008	0.006	0.005	0.004	0.001	0.007	0.007	0.009	0.002
	5	0.015	0.019	0.018	0.010	0.006	0.022	0.025	0.030	0.007
	6	0.034	0.048	0.045	0.028	0.020	0.051	0.058	0.054	0.023
	7	0.056	0.072	0.072	0.063	0.034	0.069	0.077	0.071	0.067
	8	0.054	0.059	0.060	0.060	0.035	0.055	0.053	0.058	0.066
	9	0.053	0.050	0.044	0.049	0.037	0.046	0.037	0.047	0.054
	10	0.056	0.051	0.044	0.048	0.047	0.046	0.037	0.046	0.051
	11	0.058	0.053	0.046	0.050	0.056	0.049	0.042	0.050	0.056
	12	0.060	0.054	0.049	0.052	0.055	0.052	0.045	0.053	0.071
	13	0.062	0.057	0.050	0.055	0.056	0.053	0.045	0.054	0.066
	14	0.067	0.064	0.056	0.062	0.061	0.060	0.057	0.063	0.060
	15	0.074	0.075	0.080	0.071	0.071	0.070	0.073	0.069	0.062
	16	0.080	0.083	0.088	0.083	0.080	0.077	0.087	0.072	0.063
	17	0.077	0.083	0.089	0.093	0.107	0.081	0.090	0.077	0.075
	18	0.060	0.062	0.059	0.072	0.085	0.065	0.068	0.062	0.070
	19	0.045	0.043	0.048	0.049	0.066	0.049	0.049	0.044	0.053
	20	0.036	0.034	0.036	0.040	0.060	0.039	0.040	0.035	0.044
	21	0.031	0.030	0.031	0.039	0.049	0.036	0.037	0.033	0.035
	22	0.024	0.023	0.026	0.028	0.038	0.028	0.029	0.026	0.033
	23	0.018	0.016	0.020	0.019	0.020	0.020	0.019	0.021	0.019

(continued on next page)

Table H.6. Default Hour-of-Day Factors for Passenger Cars (continued)

		Functional Class								
Category	Hour	1	2	6	7	8	11	12	14	16
Weekend	0	0.016	0.016	0.024	0.022	0.016	0.021	0.023	0.023	0.028
	1	0.011	0.010	0.013	0.012	0.009	0.014	0.015	0.014	0.023
	2	0.009	0.006	0.007	0.007	0.004	0.010	0.008	0.010	0.021
	3	0.007	0.005	0.005	0.005	0.002	0.006	0.005	0.006	0.008
	4	0.007	0.005	0.005	0.004	0.002	0.006	0.005	0.006	0.005
	5	0.010	0.009	0.008	0.006	0.002	0.010	0.009	0.010	0.005
	6	0.017	0.016	0.016	0.010	0.007	0.017	0.016	0.017	0.011
	7	0.027	0.025	0.025	0.018	0.014	0.025	0.023	0.024	0.018
	8	0.039	0.036	0.033	0.028	0.025	0.035	0.036	0.035	0.030
	9	0.052	0.050	0.045	0.039	0.046	0.047	0.045	0.046	0.048
	10	0.063	0.062	0.055	0.049	0.058	0.057	0.057	0.056	0.054
	11	0.070	0.070	0.063	0.060	0.063	0.065	0.066	0.054	0.057
	12	0.072	0.075	0.072	0.068	0.086	0.072	0.076	0.071	0.074
	13	0.072	0.075	0.073	0.071	0.081	0.072	0.073	0.071	0.071
	14	0.073	0.075	0.072	0.073	0.077	0.071	0.074	0.072	0.069
	15	0.075	0.077	0.082	0.074	0.077	0.073	0.075	0.073	0.067
	16	0.075	0.078	0.075	0.079	0.086	0.073	0.075	0.073	0.071
	17	0.071	0.075	0.074	0.080	0.086	0.072	0.071	0.073	0.068
	18	0.061	0.065	0.067	0.075	0.067	0.064	0.063	0.063	0.067
	19	0.051	0.051	0.054	0.063	0.056	0.052	0.051	0.052	0.056
	20	0.041	0.041	0.043	0.052	0.051	0.043	0.043	0.044	0.049
	21	0.033	0.033	0.035	0.045	0.039	0.038	0.037	0.038	0.040
	22	0.026	0.026	0.029	0.034	0.032	0.032	0.032	0.033	0.035
	23	0.019	0.019	0.024	0.025	0.019	0.025	0.023	0.026	0.024

are listed in Table H.8. The vehicle classes include motorcycles; passenger cars; other two-axle, four-tire, single-unit vehicles; and two-axle, six-tire, single-unit trucks. Collectively, these classes represent about 90% of the traffic stream. These factors were obtained from tables by Hallenbeck et al. (1997, pp. 65–68). The report did not provide these factors for rural minor collectors, so the factors for rural major collectors are substituted in the exhibit.

Traffic Incident Procedure

This section lists the default values for the distribution of incidents by the categories identified in the following list. The last three categories define the incident type.

- Weather condition
  - No precipitation and dry pavement
  - Rainfall
  - Wet pavement but not raining
  - Snowfall
  - Snow or ice on pavement but not snowing;
- Street location
  - Segment
  - Signalized intersection;
- Event type
  - Crash
  - Noncrash;
- Lane location
  - One lane
  - Two or more lanes
  - Shoulder; and
- Severity
  - Property-damage-only crash
  - Fatal or injury crash
  - Breakdown
  - Other.

Table H.7. Default Day-of-Week Factors for Passenger Cars

Number	Day of Week	Urban	Rural
1	Sunday	0.87	1.01
2	Monday	0.98	0.95
3	Tuesday	0.98	0.91
4	Wednesday	1.00	0.93
5	Thursday	1.03	0.98
6	Friday	1.15	1.16
7	Saturday	0.99	1.05

A review of the literature indicated that most examinations of incident data focus on freeways, and few consider urban streets. Also, very few of these examinations separately quantify incidents by weather condition. No publications were identified that separately addressed incident duration for street segments and for signalized intersections.

Weather Conditions

Data for incidents on highways in New York were examined by List et al. (2008, Table 6.17, p. 3.1-38). A total of 1,083

Table H.8. Default Month-of-Year Factors for Four Vehicle Classes Combined[a]

		Functional Class								
Number	**Month**	**1**	**2**	**6**	**7**	**8**	**11**	**12**	**14**	**16**
1	January	0.747	0.813	0.834	0.812	0.812	0.836	0.802	0.831	0.881
2	February	0.828	0.855	0.935	0.935	0.935	0.863	0.874	1.021	0.944
3	March	0.926	0.891	0.973	0.977	0.977	0.936	0.936	1.030	1.016
4	April	0.994	0.958	1.004	1.044	1.044	0.992	0.958	0.987	0.844
5	May	1.087	1.091	1.091	1.009	1.009	0.990	1.026	1.012	1.025
6	June	1.105	1.087	1.106	1.041	1.041	1.039	1.068	1.050	1.060
7	July	1.243	1.125	1.016	0.982	0.982	1.152	1.107	0.991	1.150
8	August	1.137	1.130	1.015	1.056	1.056	1.050	1.142	1.054	1.110
9	September	1.087	1.038	1.062	1.054	1.054	1.081	1.088	1.091	1.081
10	October	0.996	1.041	1.080	1.028	1.028	1.012	1.069	0.952	1.036
11	November	0.974	0.965	0.983	1.007	1.007	1.012	0.962	0.992	0.989
12	December	0.872	0.910	0.966	0.998	0.998	0.995	0.933	0.938	0.903

[a]Motorcycles, passenger cars, other two-axle four-tire single-unit vehicles, and two-axle, six-tire single-unit trucks.

incidents were identified for which weather conditions were reported. The distribution of incident type by weather condition indicated that, for any given incident type, the proportion varied less than 0.01 among weather conditions. For example, the proportion of property-damage-only crashes decreased from 0.42 for no precipitation and dry pavement to 0.41 for wet pavement, and it increased to 0.43 when snow or ice was on the pavement. This pattern was also noted by Andrey et al. (2001) following their review of weather-related safety research.

Although the trends noted in the previous paragraph are plausible, they are not based on data for urban streets, and the effect appears to be very small. Therefore, no adjustment is made to the default incident type distribution based on weather. Additional research is needed to determine the severity distribution for weather conditions.

The literature review identified two research publications that quantified the effect of weather condition on incident duration. Garib et al. (1997) examined the duration of 277 incidents occurring on I-880 in Oakland, California. They found that incident duration during rainfall was reduced by 21% relative to incidents occurring without rainfall.

The data assembled by List et al. (2008, Table 34, p. 3.2-36) were examined with regard to the influence of weather on incident duration. This examination indicated that incident duration was reduced by about 18% relative to clear or cloudy conditions when the pavement was wet but there was no precipitation. When there was precipitation, incident duration was reduced by 20% (although heavy rain was noted to increase duration). In contrast, the presence of snow or ice on the pavement tended to increase incident duration by 36%.

The findings associated with incident duration indicated a significant effect of weather condition. The percentages attributed to List et al. were used to derive the default durations by weather condition.

Default Values

Table H.9 shows the default incident type distribution and duration values for conditions with no precipitation and dry pavement. The same distribution values were used for all weather conditions and street locations. Similarly, the same duration values were used for all street locations. This approach was taken because no information could be found regarding the possible variation of these values by street location. Specifically, documented evidence regarding the variation of incident frequency or duration by street location could not be found in the literature or in the available agency incident records.

The proportions shown in Table H.9 are based on incident data collected by the SHRP 2 Project L08 research team. These data were obtained from incident logs for five arterial streets in California totaling 86.5 mi. A total of 2,081 incidents are in the database used to derive the proportions shown.

The proportions shown for lane location indicate that many incidents occur on the shoulders of the streets included in the data assembled by the SHRP 2 L08 researchers. The proportion of the streets in these data that have shoulders is unknown. Given that many urban streets do not have shoulders, the extent to which shoulder presence influenced the lane location distribution shown in Table H.9 is unclear. When shoulders are not present, the proportions allocated to the shoulder category should be added to those for the one-lane category to estimate the likely distribution. Ideally, additional research would be conducted to separately develop the lane location distribution for streets with and without shoulders.

The default joint proportion for each incident shown in Column 8 is used in the reliability methodology. The average incident duration shown in the far-right column is also used in the reliability methodology. It is equal to the sum of the incident detection time, response time, and clearance time. Research by Raub and Schofer (1997) indicates that

Table H.9. Default Incident Values for No Precipitation and Dry Pavement

Street Location	Event Type	Proportion	Lane Location	Proportion	Severity	Proportion	Joint Proportion	Response Time (min)	Clearance Time (min)	Total Time (min)
Segment	Crash	0.358	One lane	0.335	Fatal or Inj	0.304	0.036	15.0	56.4	73.4
		0.358		0.335	PDO	0.696	0.083	15.0	39.5	56.5
		0.358	2+ lanes	0.163	Fatal or Inj	0.478	0.028	15.0	56.4	73.4
		0.358		0.163	PDO	0.522	0.030	15.0	39.5	56.5
		0.358	Shoulder	0.502	Fatal or Inj	0.111	0.020	15.0	56.4	73.4
		0.358		0.502	PDO	0.889	0.160	15.0	39.5	56.5
	Non-crash	0.642	One lane	0.849	Breakdown	0.836	0.456	15.0	10.8	27.8
		0.642		0.849	Other	0.164	0.089	15.0	6.7	23.7
		0.642	2+ lanes	0.119	Breakdown	0.773	0.059	15.0	10.8	27.8
		0.642		0.119	Other	0.227	0.017	15.0	6.7	23.7
		0.642	Shoulder	0.032	Breakdown	0.667	0.014	15.0	10.8	27.8
		0.642		0.032	Other	0.333	0.007	15.0	6.7	23.7
						Total:	1.000			
Inter-section	Crash	0.310	One lane	0.314	Fatal or Inj	0.378	0.037	15.0	56.4	73.4
		0.310		0.314	PDO	0.622	0.061	15.0	39.5	56.5
		0.310	2+ lanes	0.144	Fatal or Inj	0.412	0.018	15.0	56.4	73.4
		0.310		0.144	PDO	0.588	0.026	15.0	39.5	56.5
		0.310	Shoulder	0.542	Fatal or Inj	0.109	0.018	15.0	56.4	73.4
		0.310		0.542	PDO	0.891	0.150	15.0	39.5	56.5
	Non-crash	0.690	One lane	0.829	Breakdown	0.849	0.486	15.0	10.8	27.8
		0.690		0.829	Other	0.151	0.086	15.0	6.7	23.7
		0.690	2+ lanes	0.141	Breakdown	0.865	0.084	15.0	10.8	27.8
		0.690		0.141	Other	0.135	0.013	15.0	6.7	23.7
		0.690	Shoulder	0.030	Breakdown	0.875	0.018	15.0	10.8	27.8
		0.690		0.030	Other	0.125	0.003	15.0	6.7	23.7
						Total:	1.000			

the detection time varies from 1 to 2 min. A default value of 2.0 min is used in the reliability methodology described in this paper.

The average response times listed in Table H.9 are shown to be 15 min for all incident times. It is likely that this time will vary among jurisdictions and facilities, depending on the priority placed on street system management and the connectivity of the street system. Dowling et al. (2004) indicate this time can vary from 5 to 30 min for freeways, with the shorter time likely when freeway service patrols are used. A default value of 15 min is used for all weather conditions, except when snow is on the pavement. When there is snowfall, or snow or ice on the pavement, this value is increased 36% (20.4 min) based on the analysis discussed in the section above titled "Weather Conditions." Additional research is needed to quantify this time by incident type and street location.

The average clearance times shown in Column 10 of Table H.9 are based on times reported by Raub and Schofer (1997). They are based on an evaluation of 1,497 incidents on urban streets in Illinois. The durations reported by Raub and Schofer equal the sum of the response time and clearance time. A response time of 15 min was subtracted from the reported durations to obtain the clearance times shown in the exhibit. The times reported for disabled and fire were combined to obtain the values shown for breakdown.

The clearance times shown in Table H.9 are consistent with those found by the SHRP 2 L08 researchers in their examination of clearance times for several arterial streets in California and Oregon. One exception to this consistency is with the noncrash incidents in California. The clearance times for noncrash incidents in California are as long, or longer, than those for crash-related incidents. It is believed that these longer clearance times reflect the occasional occurrence of road closure by landslide, which is not representative of most streets in the United States.

The times shown in Column 10 of Table H.9 are adjusted for weather conditions based on the analysis discussed above in "Weather Conditions." Specifically, for rainfall conditions, the default clearance time is reduced such that the combined response time and clearance time is decreased by 20%. When there is wet pavement but no rainfall, the default clearance time is reduced such that the combined response time and clearance time is decreased by 18%. When there is snowfall or snow or ice on the pavement (but no snowfall), the default clearance time is increased by 36%.

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APPENDIX I

Example Problem: Existing Freeway Reliability

Objective

This example problem illustrates the following process:

1. Calculating reliability statistics for a freeway facility using the minimum required data for the analysis;
2. Identifying key reliability problems on the facility; and
3. Diagnosing the causes (e.g., demand, weather, incidents) of reliability problems on the facility.

Site

The study freeway facility is a 12.5-mi portion of eastbound I-40 between Durham and Raleigh, North Carolina, bounded by NC-55 to the west and NC-54 to the east (see Figure I.1). The eastbound direction is most heavily used by commuters on weekdays, with a peak hour of 5 to 6 p.m. The posted speed limit is 65 mph. A weaving section near the downstream end of the facility creates a recurring bottleneck.

Minimum Required Data Inputs

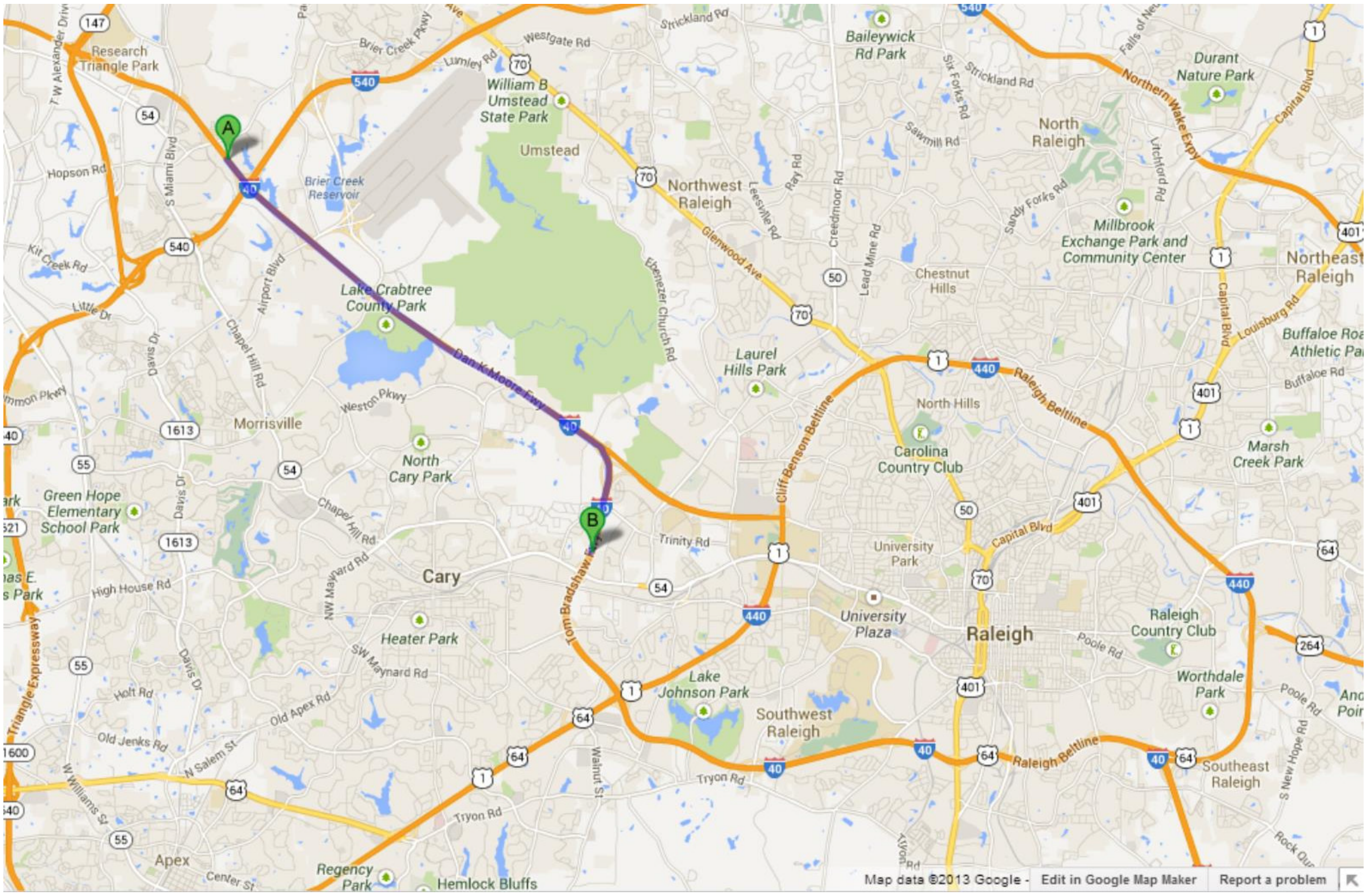
The data listed below are required to perform a reliability analysis of a freeway facility. Additional desirable data are also identified, but this example problem assumes that the additional desirable data are not available. Instead, this example illustrates the use of defaults and lookup tables to substitute for the desirable data.

- Data required for a *2010 Highway Capacity Manual* (HCM2010) (Transportation Research Board of the National Academies 2010) freeway facility analysis (Chapter 10):
    - Facility volumes by 15-min analysis periods (time slices) for a single day's peak period
    - Desirable: single day's peak period facility travel times for calibrating a traditional HCM2010 operations analysis model for the facility
    - Facility geometry and controls by analysis segment and by analysis period (if controls vary by analysis period) for the study period (if controls or geometry vary by time of day, day of week, or month of year);
- Data required to estimate demand variability:
    - Annual average daily traffic (AADT), directional factor (*D*), and peak period demand profiles (*K*-factors)
    - Desirable: archived peak period mainline volume counts for previous year;
- Data required to estimate incident frequencies:
    - Collision reports for the prior 3-year period
    - Desirable: detailed incident logs including frequency, duration, and location of incidents for a similar period;
- Data required to estimate weather frequencies:
    - Weather reports for at least the prior 3-year period
    - Desirable: 10-year weather data from a nearby weather station; and
- Optional extra data for calibrating estimates:
    - Facility travel times (or spot speeds) and volumes by 15-min analysis periods (time slices) for the target study period (peak periods, days of weeks, months of year, and so forth).

Computational Steps

This example problem proceeds through the following steps:

1. Scoping the bounds of the reliability analysis:
    a. Establishing the analysis purpose, scope, and approach
    b. Selecting an appropriate study period
    c. Selecting an appropriate reliability reporting period
    d. Selecting appropriate reliability performance measures and thresholds of acceptable performance;
2. Coding the HCM facility operations analysis:
    a. Identifying the sources of unreliability to be analyzed
    b. Coding base conditions
    c. Coding alternative data sets, if any;



Source: © 2013 Google.

Figure I.1. Study freeway facility bottleneck during peak demand levels.

3. Estimating the demand variability profile;
4. Estimating severe-weather frequencies;
5. Estimating incident frequencies;
6. Generating scenarios and the probabilities of their occurrence;
7. Applying the HCM2010 freeway facility analysis method;
8. Performing quality control and error checking and determining inclusion thresholds;
9. Calculating performance measures; and
10. Interpreting results.

Step 1. Scoping the Bounds of the Reliability Analysis

Although most professional engineers and planners are already well trained in scoping a traditional highway capacity analysis, travel time reliability introduces some extra considerations that are not part of a traditional capacity analysis:

- Selecting an appropriate study period for reliability (hours of day) and an appropriate reliability reporting period (days of week, months of year);
- Selecting appropriate reliability performance measures according to the agency's reliability objectives and the facility type; and
- Selecting thresholds of acceptable performance.

A reliability analysis has much greater data and computational demands than a traditional HCM operations analysis. Therefore, it should be tightly scoped to ensure the analyst has the resources to complete the analysis. Furthermore, a loosely scoped analysis that provides more days and hours than needed runs the risk of diluting the reliability results by mixing in too many hours or days of free-flow conditions into the analysis.

Purpose

To focus the analysis, it is important to identify the purpose for performing the reliability analysis. In this example, the purpose of performing the reliability analysis of existing conditions is to

- Determine if the facility is experiencing significant reliability problems; and

- Diagnose the primary causes of the reliability problems on the facility so that an improvement program can be developed for the facility.

Determining the Reliability Analysis Box

The reliability reporting period has three dimensions: (1) the geometric limits of the facility to be evaluated (the study section), (2) the periods within the day when the analysis is to be performed (the study period), and (3) the days of the year over which reliability is to be computed and reported (the reliability reporting period). The result is a spatial–temporal cube (see Figure I.2) within which reliability is computed.

The reliability box should be dimensioned so that it includes all the recurring congestion (congestion occurring under recurring demand conditions, in fair weather, without incidents) of interest for the analysis. This requirement favors a large reliability box. However, the larger the reliability box, the greater the number of instances of free-flow conditions, which will tend to mask or dilute the reliability problems.

In this example, an examination of the facility over several days determined the general spatial and temporal boundaries of congestion on the facility under fair weather, nonincident conditions. The selected study period was the 6-h-long weekday afternoon peak period (2 to 8 p.m.), and the study section was a 12.5-mi facility length between NC-55 and NC-54 (corresponding to 34 HCM analysis segments). All the instances when speeds regularly dropped below 40 mph are encompassed within the selected study section and study period. Figure I.2 shows an example of the speed profile generated by FREEVAL-RL when an incident occurs in the furthest downstream segment on the facility.

Once the study section length and study period have been selected, the next step is to determine for how many (and which) days of the year reliability will be computed (the reliability reporting period). The objective of setting the reliability reporting period is to focus the analysis on days when reliability is a concern. The reporting period should include enough days so that the probability of encountering a significant number and range of incident types is high. A minimum of 100 days is recommended for the reporting period, although a full-year analysis is preferred.

Thus, for this example, weekdays for a full year were selected for the reliability reporting period. At five weekdays per week, 52 weeks plus 1 day per year, there are 261 weekdays per year (including holidays). Holidays may be excluded from the reliability reporting period if they result in lower than normal p.m. peak period demands. (In this case, holidays were not deemed to be a significant factor affecting reliability, and were therefore included in the reliability analysis.)

If an agency wishes to focus on nonweather effects and avoid vacation effects, then a single season may be selected, rather than a full year. The selection of the appropriate reliability reporting period hinges on the agency's purpose for the analysis.

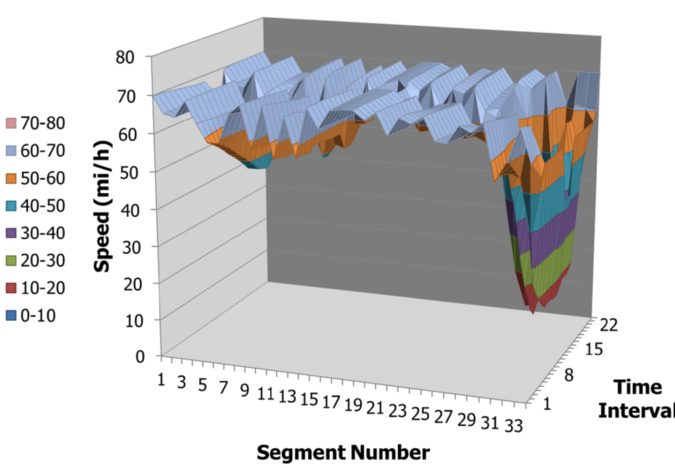


Figure I.2. Sample congested speed profile on I-40.

Selecting Reliability Performance Measures

For instructional purposes, all the reliability performance measures shown in Table I.1 will be computed. However, for a typical application, one or two performance measures most useful to the agency's analysis purpose are recommended to be selected.

Since all performance measures are derived from the same travel time distribution, once an agency has picked one or two measures for the reliability analysis, additional measures do not bring significant new information to the results. In that sense, it is most important that an agency selects performance

Table I.1. Reliability Performance Measures to Be Evaluated

Measure	Definition
Mean TTI	Mean travel time divided by free-flow travel time
Planning time index	95th percentile travel time divided by free-flow travel time
80th percentile TTI	80th percentile travel time divided by free-flow travel time
Semistandard deviation	One-sided standard deviation, referenced to free-flow
Failure/on-time	Percent of trips less than 40 mph
Standard deviation	Usual statistical definition
Misery index	Average of top 5% of travel times divided by free-flow travel time
Reliability rating	Percentage of vehicle miles traveled at a TTI less than 1.33

measures consistently across different reliability analyses, allowing agency staff and stakeholders to begin developing an understanding of these metrics.

In this example, the agency could pick the mean travel time index (TTI) so that average performance could be evaluated (the mean is useful for computing total benefits later). As an indicator of reliability, the agency could pick the 80th percentile TTI or the planning time index (PTI).

Selecting Thresholds of Acceptable Performance

Ideally, an agency has already developed its own thresholds of acceptable reliability performance based on locally collected data. However, in this case, the agency responsible for the freeway has not yet assembled sufficient data on the reliability of its own facilities to have confidence in setting its own standards. Consequently, two standards of performance will be evaluated in this example problem as part of the reliability assessment.

The first standard will be determined by comparing the performance of the I-40 facility to other facilities in the SHRP 2 Project L08 data set. For example, the operating agency may select a performance threshold to be more reliable than the worst 10% of U.S. urban freeway facilities studied for this project. Thus, if the mean TTI for the facility is computed to be greater than 1.93, then the facility's reliability will be considered unacceptable. Similarly, if the computed PTI exceeds 3.55, that will also be considered unacceptable.

The second standard is set based on the agency's congestion management goal of operating its freeways at 40 mph or better during the majority of the peak periods within the year. This particular standard requires that a modified travel time performance index, called the policy index, be computed that uses the agency's 40-mph target speed in place of the free-flow speed.

$$PI = \frac{\text{mean travel time}}{\text{travel time at 40 mph}}$$

Since the agency's goal is for the mean annual peak period speed on the facility to be 40 mph or higher, then if the policy index exceeds 1.00, the reliability of the facility will be considered unacceptable.

Step 2. Coding the HCM Facility Operations Analysis

Selecting Reliability Factors for Evaluation

The major causes of travel time reliability problems are demand surges, weather, incidents, special events, and work zones. Evaluating all possible causes of reliability puts a significant strain on analytical resources, so it is recommended that rarer causes of unreliability be excluded from the reliability analysis. In addition, the purpose of the analysis may suggest that some causes can be bundled together.

The study facility in this case is large, and adjacent special event generators do not significantly affect operations during the selected study period (most events are on weekends). Consequently, the effects of special events do not need to be evaluated separately and can be bundled in with other causes of surges in demand. Similarly, work zones are not planned during weekday peak periods on the facility in the analysis year, so work zones can be excluded from the reliability analysis.

Coding Base Conditions

The base HCM analysis input file (the seed file) was coded for the selected study section and study period using the procedures and guidance contained in HCM2010 Chapters 10 to 13. Demands, geometries, and free-flow speed were obtained for a single, typical, fair weather, nonincident, nonholiday, weekday p.m. peak period (2 to 8 p.m.). Figure I.3 shows the geometry of the study section of the facility. Table I.2 shows a portion of the input entries for the seed file.

Mainline volumes were obtained from side-fire radar stations spaced roughly 1.5 mi apart. Ramp volumes were counted for 2 weeks using portable tube counters. A typical fair-weather weekday when daily traffic was close to the AADT was selected from the 2-week count period. Default values of 5% trucks, 0% recreational vehicles, and 0% buses were used to account for heavy vehicles.

There were no extended grades in excess of 2% for longer than 0.5 mi on the facility (see HCM2010, p. 11–15), and the facility has a generally level vertical profile, so a general terrain category of level was used to characterize the vertical geometry of the facility.

Segment lengths and number of lanes were obtained by field inspection or Google aerial photos. Lane widths are a standard 12 ft. The free-flow speed was estimated using HCM2010 Equation 11-1.

Coding Alternative Data Sets

As there is no need to account for special events or work zones, no alternative data sets need to be created. If there had been a need for them, they would have been developed in the same way as the base data set, with appropriate modifications to the input data to reflect changes in demand, geometry, and traffic control.

Step 3. Estimating the Demand Variability Profile

The total number of scenarios that must be evaluated significantly affects the processing time and the time required by the

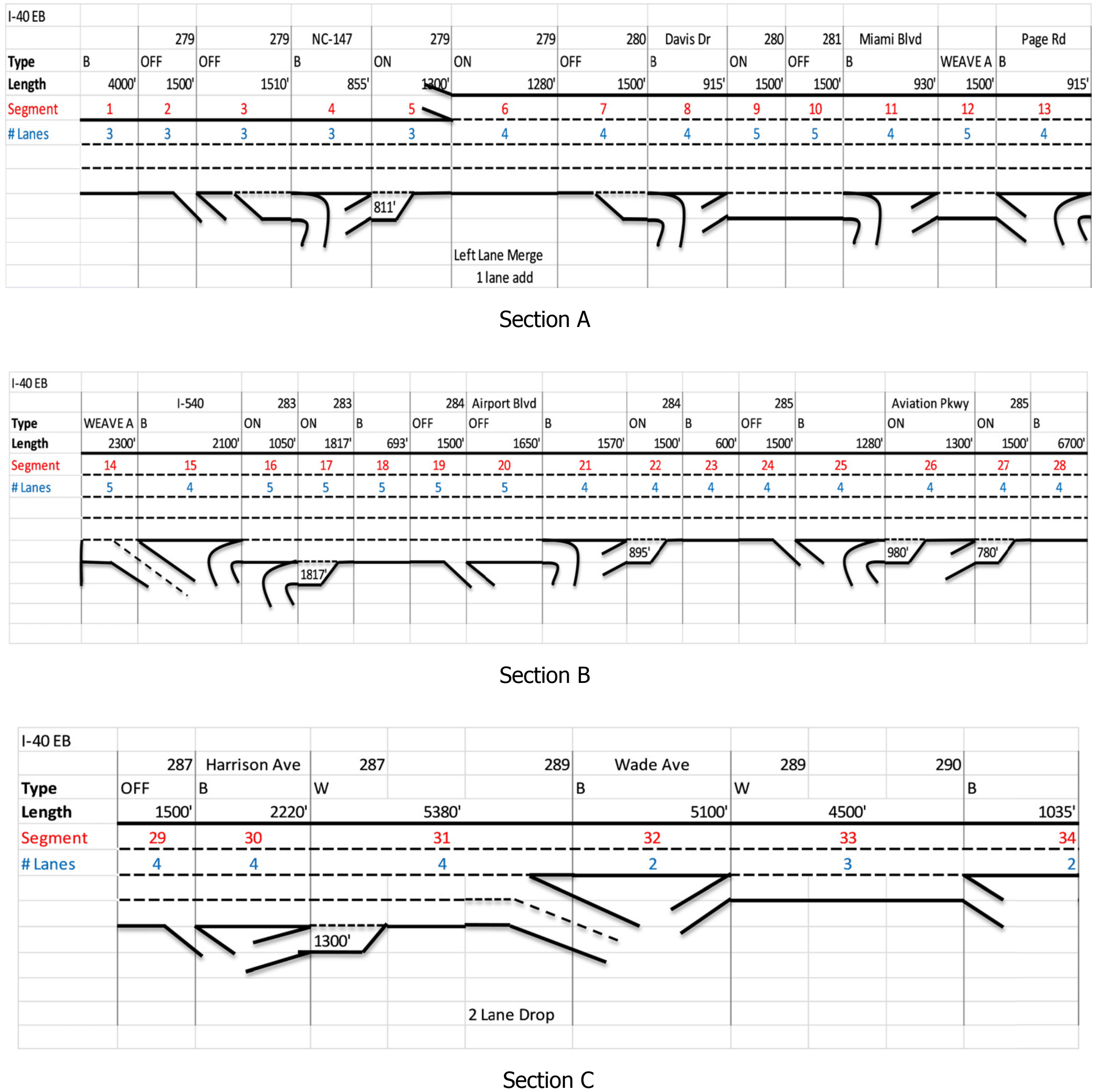


Figure I.3. Geometry of facility study section.

analyst to analyze the results. The number of scenarios is the product of the number of demand levels, weather levels, and incident levels selected for evaluation. Thus, any reduction in the number of unnecessary demand, weather, and incident levels needed for the reliability analysis will result in significant processing and evaluation time savings.

An examination of local data on I-40 demand variability over the course of a year (see Table I.3) revealed that weekday demand variability over the year at the site could be adequately represented by three demand patterns (Monday to Wednesday, Thursday, and Friday) and four month types grouped by the major seasons of the year (December to February; March to May; June to August; and September to November). Thus it was possible to consolidate 60 potential demand levels (five weekday times 12 months) into 12 demand levels (three weekday patterns by four month types). Days and months with similar ratios of monthly average daily traffic (ADT) to AADT for a given demand pattern were grouped together. All entries were normalized to a Monday in January.

Entries in Table I.3 are ADT demand adjustments for a given combination of day and month relative to ADT for a Monday in January. Table I.4 shows the consolidated table of demand ratios for the example problem, and Table I.5 shows the percentage time of year by season and demand pattern.

Step 4. Estimating Severe-Weather Frequencies

Exhibit 10-15 in HCM2010 identifies five weather types (rain, snow, temperature, wind, and visibility) with varying intensity

Table I.2. Sample Freeway Input Entries for Seed File

Input Worksheet - Directional Freeway Facility	Release May 9th, 2012							
FREEWAY SYSTEM TITLE:	I-40							
SEGMENT NUMBER :	1	2	3	4	5	6	7	8
SEGMENT LABEL :	S01	147S	147N	S04	147N	147S	S07	Davis
Type (B, ONR, OFR, R, or W)	B	OFR	OFR	B	ONR	ONR	R	OFR
Length (ft)	4000	1500	1500	855	1300	1280	220	1280
Number of Lanes	3	3	3	3	3	3	4	4
FF Speed (Mi/hr)	70	70	70	70	70	70	70	70
Segment Demand (vph)	3,427	3,427	3,359	3,017	3,395	4,889	4,889	4,889
Vehicle Occupancy (pass/veh)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Capacity Adjustment Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Origin Demand Adjustment Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Destination Demand Adjustment Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
FF Speed Adjustment Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Lane Width (ft)	12	12	12	12	12	12	12	12
Lateral Clearance (ft)	4	4	4	4	4	4	4	4
% Trucks	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
% RV's	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Terrain	level	level	level	level	level	level	level	level
Truck Passenger Car Equivalent ET	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
R.V. Passenger Car Equivalent ER	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
On-Ramp Demand (vph)					379	1,493		
On-Ramp % Trucks					5	5		
On-Ramp % RV's					0	0		
Off-Ramp Demand(vph)		68	342					190
Off-Ramp % Trucks		5	5					5
Off-Ramp % RV's		0	0					0
Acc/ Dec Lane Length (ft)		300	300		800	1280		300
Number of Lanes on Ramp		1	1		1	1		1
Ramp on Left or Right (L / R)		Right	Right		Right	Left		Right
Ramp FFS (mi/hr)		45	45		45	55		45
Ramp Metering Rate (vph)					2100	2100		
Ramp-to-Ramp Demand (vph)								

Table I.3. Demand Ratios for I-40 Case Study (ADT for Mondays in January)

Month	Day of Week				
	Monday	Tuesday	Wednesday	Thursday	Friday
January	1.00	1.03	1.04	1.05	1.08
February	0.94	1.01	1.04	1.09	1.14
March	1.04	1.07	1.06	1.11	1.17
April	1.07	1.09	1.10	1.16	1.22
May	1.08	1.11	1.11	1.16	1.21
June	1.08	1.09	1.07	1.14	1.18
July	1.08	1.07	1.10	1.15	1.18
August	1.05	1.05	1.06	1.09	1.16
September	1.02	1.02	1.02	1.07	1.15
October	1.05	1.05	1.07	1.11	1.16
November	0.97	1.00	1.04	1.08	1.07
December	0.97	0.96	0.99	0.92	1.01

Table I.4. Consolidated Demand Ratios for I-40 Case Study

Season	Monday–Wednesday	Thursday	Friday	Average
Winter	0.9969	1.0202	1.0765	1.0398
Spring	1.0813	1.1435	1.1989	1.0443
Summer	1.0689	1.1264	1.1767	1.0916
Fall	1.0267	1.0878	1.1281	1.1272
Average	1.0435	1.0945	1.1450	1.0744

Table I.5. Time of Year by Season and Demand Pattern

Season	Monday–Wednesday (%)	Thursday (%)	Friday (%)	Average (%)
Winter	13.903	4.887	5.255	24.045
Spring	15.179	4.933	4.933	25.045
Summer	15.475	5.022	5.022	25.519
Fall	15.246	5.066	5.079	25.391
Average	59.804	19.907	20.289	100.000

levels that affect the capacity of freeways. Some of these categories or intensity levels have a negligible effect on freeway capacities (4% or less effect) and are consequently neglected in the reliability analysis. Based on this criterion, rain under 0.10 in./h, temperature events above –4°F, and all wind events are consolidated into the nonsevere weather category because of their negligible effects on capacity. A 10-year weather history of National Weather Service meteorological aviation report data was obtained for the nearby Raleigh–Durham Airport from Weather Underground (http://www.wunderground.com/history/).

The data were filtered to eliminate unknown (-9999) conditions. The time between reports was calculated to obtain the duration of each weather report and to account for missing reports. The data were then classified into the categories defined in Table I.6.

The percentage of time during the reliability reporting period that each of the weather categories are present was computed by dividing the total number of minutes for each weather category observed in the prior 10 years during the reliability reporting period by the total number of minutes within the reliability reporting period (Table I.6). The total number of minutes within the reliability reporting period for the 10-year period of weather observations (939,600 min) was computed for this example by multiplying the 6-h study period per day by 60 min per hour by 261 weekdays per year (five weekdays per week times 52 weeks per year plus 1 day) by 10 years. In cases for which multiple weather categories are present (e.g., poor visibility during a snow event), the most severe condition (the one most affecting capacity) is assumed to control, and the event is assigned to that weather category.

Table I.6. Presence of Weather Categories on I-40 by Percentage Time per Month

Month	Rain		Snow				Severe Cold (%)	Visibility			Nonsevere Weather (%)
	Med. (%)	Heavy (%)	Light (%)	Light–Med. (%)	Med.–Heavy (%)	Heavy (%)		Low (%)	Very Low (%)	Min. (%)	
January	1.97	0.00	5.91	0.00	0.00	0.00	0.00	0.00	0.00	0.00	92.12
February	2.72	0.00	0.00	0.00	0.00	0.00	0.00	2.17	0.00	0.00	95.11
March	0.51	0.00	1.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	98.48
April	0.00	0.54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	99.46
May	1.95	1.95	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	96.10
June	0.51	0.51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	98.99
July	0.50	0.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	99.00
August	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	100.00
September	4.26	0.53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	95.21
October	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	100.00
November	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	100.00
December	0.00	0.00	7.81	0.49	0.00	0.00	0.00	0.00	0.00	0.00	91.71
Year	1.03	0.34	1.23	0.04	0.00	0.00	0.00	0.18	0.00	0.00	97.18

Note: Med. = medium; Min. = minimal.

Table I.7. Estimated Percentage of Time Weather Events Present on I-40 by Season

Season	Medium Rain (%)	Heavy Rain (%)	Light Snow (%)	Light–Med. Snow (%)	Low Visibility (%)	Normal Weather (%)	Total (%)
Winter	1.496	0.000	4.745	0.175	0.679	92.905	100.000
Spring	0.797	0.802	0.352	0.000	0.000	98.049	100.000
Summer	0.335	0.335	0.000	0.000	0.000	99.330	100.000
Fall	1.440	0.180	0.000	0.000	0.000	98.380	100.000
Total	1.010	0.332	1.229	0.042	0.163	97.223	100.000

Entries are minutes of identified weather type divided by total minutes of weekday study periods (in this example, weekdays, 6-h p.m. peak) for that month. Monthly and annual percentages total to 100% for each month and for the full year.

Weather categories with less than 0.1% probability for a given month in the 10-year weather history were dropped from further consideration to manage the number of scenarios. Based on this criterion, severe cold, medium-heavy and heavy snow, and very low and minimal visibility were dropped, and the probabilities of all remaining categories renormalized to add up to 100%. The final set of six weather categories and intensity levels selected for this example problem are shown in Table I.7 along with their estimated probabilities.

Seasonal weather probabilities are assumed to apply identically to all demand patterns within the season, and weather is assumed to be independent of demand pattern within the season.

Step 5. Estimating Incident Frequencies

Exhibit 10-17 in HCM2010 identifies the capacity effects of five incident types (shoulder disablement, shoulder accident, one lane blocked, two lanes blocked, and three lanes blocked). The shoulder disablement category was dropped for this example problem because its capacity effects are 1% for facilities with three or more lanes, such as the facility in this example.

The HCM analysis method, like all methods limited to a single facility, cannot produce meaningful results for complete facility closures, since any methodology confined to a single facility cannot predict demand rerouting to other facilities. Therefore, the evaluation of incidents in this example is limited to incidents that maintain at least one lane open to traffic. The facility is mostly four lanes in one direction, but there are some segments with only two or three lanes.

In this example, generalized crash data were available, but reliable incident logs that indicated incident type by number of lanes closed were not. Five years of crash data were obtained for the 12.5-mi-long eastbound direction of I-40. The data indicated that this portion of I-40 experiences an average of 164.5 crashes per 100 million vehicle miles traveled (VMT).

The crash rate for this facility was then expanded to incidents by lane and shoulder closure type by using an expansion factor. A local study comparing shoulder and lane closure incidents to reported crashes found that there were approximately seven incidents involving shoulder or lane closures for every reported crash on I-40.

The expected number of incidents I by month m for the facility is computed as shown in Equation I.1:

$$I(m) = \frac{\text{CR} \times \text{ICR} \times \text{VMT}(\text{seed}) \times \text{DM}(m)}{100 \times 10^6 \times \text{SFDM}} \qquad (\text{I.1})$$

where

I_m = expected number of incidents in month m in the subject direction of travel;
CR = reported crash rate, crashes per 100 million VMT;
ICR = ratio of incidents to reported crashes, incidents/crash;
VMT(seed) = seed file VMT on facility in subject direction during study period, VMT;
DM(m) = demand multiplier for month m; and
SFDM = seed file demand multiplier, the ratio of seed file study period demand to AADT for the study period.

The estimated number of incidents is split into severity types and mean durations by using the values shown in Table I.8.

Finally, the probability of an incident type is computed as shown by Equation I.2:

$$p(t,m) = 1 - e^{-(I(m) \times P(t) \times D(t)/\text{SP})} \qquad (\text{I.2})$$

where

$p(t,m)$ = probability that incident type t is present in month m;
$I(m)$ = expected number of incidents in subject direction in month m;
$P(t)$ = proportion of incidents of type t;
$D(t)$ = mean duration of incidents of type t, min; and
SP = study period duration, min.

Table I.8. Mean Duration and Distribution of Incidents by Severity

Severity	Shoulder Closed	One Lane Closed	Two Lanes Closed	Three or More Lanes Closed	Total
Mean incidents (%)	75.4%	19.6%	3.1%	1.9%	100.0%
Mean duration (min)	34.0	34.0	53.6	69.6	35.4[a]

[a] Average weighted by the relative frequencies.

The resulting estimated average percentage time with incidents present on the facility is shown in Table I.9. Results that are specific to individual demand patterns are too numerous to show here.

The entries in Table I.9 represent the probability of having a given incident type in each month. The values were computed using a crash rate of 164.5 per 100 million VMT, a rounded crash-to-incident expansion factor of 7, and a seed VMT of 330,006 in Equation I.2. Incidents were computed using Equation I.1. Monthly and annual values total to 100% for each demand pattern.

Step 6. Generating Scenarios and the Probabilities of Their Occurrence

Base Scenario Development

The base scenario represents a specific combination of a demand level, a weather type, and an incident type. The demand levels are specified by month and day of week rather than by volume level. This specification enables the analyst to partially account for the effects of demand on incidents, and the effects of weather on demand, by using calendar-specific weather and incident probabilities.

The initial estimate of the percentage time that each scenario represents of the reliability reporting period is the product of the demand, weather, and incident type percentage times that combine to describe the scenario, as shown by Equation I.3. The assumption is that the percentage time of incidents and the percentage time of weather are a function of the calendar month and that other correlations between demand, incidents, and weather can be neglected.

$$\mathrm{PT}(d,w,t) = \mathrm{PT}(d) \times \mathrm{PT}(w|d) \times \mathrm{PT}(t|d) \qquad \text{(I.3)}$$

where

$\mathrm{PT}(d,w,t)$ = percentage time associated with demand pattern d with weather type w and incident type t;

Table I.9. Estimated Percentage of Time Incidents Present on I-40 Eastbound

Month	Incident Type					
	No Incident (%)	Shoulder Closed (%)	One Lane Closed (%)	Two Lanes Closed (%)	Three Lanes Closed (%)	Four Lanes Closed (%)
January	66.42	23.30	7.06	1.79	1.43	0.00
February	66.36	23.34	7.08	1.79	1.43	0.00
March	65.10	24.18	7.36	1.87	1.49	0.00
April	63.79	25.05	7.66	1.94	1.56	0.00
May	63.87	25.00	7.64	1.94	1.55	0.00
June	64.53	24.56	7.49	1.90	1.52	0.00
July	64.10	24.85	7.59	1.93	1.54	0.00
August	65.30	24.04	7.32	1.86	1.48	0.00
September	65.97	23.60	7.17	1.82	1.45	0.00
October	65.04	24.22	7.38	1.87	1.50	0.00
November	66.79	23.05	6.98	1.77	1.41	0.00
December	68.56	21.86	6.59	1.67	1.33	0.00

PT(d) = percentage time of demand pattern d within the reliability reporting period;
PT($w|d$) = percentage time of weather type w associated with demand pattern d; and
PT($t|d$) = percentage time of incident type t associated with demand pattern d.

Table I.10 shows the initial estimated scenario percentage times before the details as to starting time, location, and duration of incidents and weather have been specified. This table shows the results for only normal weather conditions. Similar computations and results are obtained for the other weather conditions. Note that the initial probabilities for all weather and incident conditions must sum to the percentage time for each demand pattern within each season.

For computing percentage time of incident type t associated with demand pattern d, the probabilities presented in Table I.10 are averaged and weighted by the number of days each demand pattern has in the calendar.

All entries are percentage time within the reliability reporting period when the specified conditions are present on the facility. Not shown are percentages for rain, snow, and low-visibility conditions. Percentages are computed using Equation I.3 and percentages from Table I.6, Table I.8, and Table I.10.

Table I.11 shows the final estimated scenario probabilities for the scenarios involving nonsevere weather. Not shown are similar tables for rain, snow, and low-visibility conditions used to derive the severe-weather column.

Specifying Incident and Weather Scenario Details

The incident starting time, duration, and location must be specified for incident scenarios. To ensure that a representative cross section of performance results are obtained, each incident scenario involving a closure of some kind is subdivided into 18 possible subscenarios (two start times, three locations, and three durations):

- Start at the beginning or the middle of the study period;
- Located at the beginning, middle, or end of the facility; and
- Enduring for the 25th, 50th, or 75th percentile highest duration for a given incident type.

Note that some subscenario options may be prohibited. For example, if the beginning, middle, or end of the facility only has three lanes, then the three-lane closure scenario is not modeled for this condition. In this case, the subscenario is removed from the total list of scenarios, and subscenarios and the probability for the removed subscenario are assigned proportionally to the remaining subscenarios.

Each of the 18 incident subscenarios is considered equally probable within the base incident scenario. Thus, each subscenario is given one-eighteenth the probability of the base scenario for the incident type.

For example, the scenario associated with Demand Pattern 1 (Mondays to Wednesdays in winter) with nonsevere weather and a shoulder closure has a 4.00645% probability of occurrence. Thus, the subscenario associated with the incident

Table I.10. Percentage Times for Incident Scenarios in Nonsevere Weather

Season	Day	No Incident (%)	Shoulder Closure (%)	One Lane Closed (%)	Two Lanes Closed (%)	Three Lanes Closed (%)	Subtotal Nonsevere Weather (%)	Subtotal Severe Weather (%)	Total (%)
Winter	M–W	8.847	3.005	0.909	0.230	0.184	13.176	1.000	14.176
	Thu	3.110	1.053	0.319	0.081	0.064	4.626	0.355	4.981
	Fri	3.344	1.135	0.343	0.087	0.070	4.979	0.385	5.364
Spring	M–W	9.660	3.710	1.132	0.287	0.230	15.019	0.307	15.326
	Thu	3.139	1.210	0.369	0.094	0.075	4.887	0.094	4.981
	Fri	3.139	1.210	0.369	0.094	0.075	4.887	0.094	4.981
Summer	M–W	9.848	3.724	1.135	0.288	0.230	15.226	0.100	15.326
	Thu	3.196	1.212	0.370	0.094	0.075	4.946	0.035	4.981
	Fri	3.196	1.212	0.370	0.094	0.075	4.946	0.035	4.981
Fall	M–W	9.702	3.468	1.053	0.267	0.213	14.704	0.239	14.943
	Thu	3.224	1.155	0.351	0.089	0.071	4.889	0.092	4.981
	Fri	3.232	1.161	0.353	0.089	0.072	4.907	0.074	4.981
Total	All	63.637	23.255	7.073	1.794	1.434	97.194	2.806	100.000

Note: M = Monday; W = Wednesday; Thu = Thursday; Fri = Friday.

Table I.11. Estimated Incident Scenario Probabilities After Adjustment

		Nonsevere Weather					Weather Subtotals		
Season	**Day**	**No Incident (%)**	**Shoulder Closed (%)**	**One Lane Closed (%)**	**Two Lanes Closed (%)**	**Three Lanes Closed (%)**	**Nonsevere (%)**	**Severe (%)**	**Total (%)**
Winter	M–W	0.008	4.006	3.637	1.373	0.871	9.896	4.28	14.176
	Thu	0.027	1.404	1.274	0.481	0.305	3.491	1.49	4.981
	Fri	0.018	1.513	1.374	0.519	0.329	3.753	1.61	5.364
Spring	M–W	0.431	4.947	4.529	1.706	1.083	12.695	2.63	15.326
	Thu	0.153	1.614	1.478	0.557	0.354	4.155	0.83	4.981
	Fri	0.153	1.614	1.478	0.557	0.354	4.155	0.83	4.981
Summer	M–W	0.581	6.384	4.541	1.721	1.098	14.324	1.00	15.326
	Thu	0.161	2.078	1.478	0.560	0.357	4.634	0.35	4.981
	Fri	0.161	2.078	1.478	0.560	0.357	4.634	0.35	4.981
Fall	M–W	0.167	5.946	4.213	1.591	1.012	12.929	2.01	14.943
	Thu	0.206	1.732	1.403	0.529	0.336	4.206	0.78	4.981
	Fri	0.087	1.991	1.411	0.533	0.339	4.361	0.62	4.981
Total	All	2.154	35.305	28.293	10.687	6.795	83.235	16.77	100.00

starting at the beginning of the study period, in the middle segment, and for an average duration will have a 4.00645%/18 = 0.22258% probability of occurrence.

The starting time and duration must also be specified for the severe-weather scenarios (e.g., rain, snow). Weather is assumed to apply equally across the entire facility. To ensure that a representative cross section of performance results is obtained, each severe-weather scenario is subdivided into two possible subscenarios: severe weather beginning at the start of the study period and severe weather beginning in the middle of the study period.

Each weather subscenario for each severe-weather base scenario is given one-half the probability of the base scenario for the weather type. For example, the scenario associated with Demand Pattern 1 (Mondays to Wednesdays in winter), with light snow and no incident, has a 0.22294% probability of occurrence. Therefore, the subscenario associated with the weather event starting at the beginning of the study period will have a 0.22294%/2 = 0.11147% probability of occurrence.

Removal of Improbable and Infeasible Scenarios

Theoretically, the procedure can generate up to 22,932 scenarios and subscenarios for the subject facility. Many of these may have exceptionally low or near-zero probability. In addition, some may be infeasible—for example, a two- or three-lane closure on a two-lane freeway segment. For this example, the improbable and zero-probability scenarios or subscenarios were removed from the reliability analysis. These exclusions translate to an inclusion threshold of near zero, meaning that all scenarios with probability greater than zero are included in the analysis. This inclusion threshold left 2,058 scenarios to be used in evaluating travel time reliability for the I-40 facility. Table I.12 shows the final scenario categorization.

It should be noted that the percentages shown here are *not the probabilities of occurrence.* They indicate the proportionate number of HCM analyses that will be performed on each scenario type for the reliability analysis. This is because each 6-h study period for incident and weather scenarios contains many 15-min analysis time periods characterized by fair weather and no incident conditions. The numbers shown in Table I.12 ensure that the initial incident and weather probabilities are honored.

Table I.12. Final Scenario Categorization

Scenario Type	**No. of Scenarios and Subscenarios**	**Total (%)**
No incidents and nonsevere weather	12	0.6
No incidents and severe weather	66	3.2
Incidents and nonsevere weather	528	25.7
Incidents and severe weather	1,452	70.6
Total	2,058	100.0

Table I.13. CAFs and Free-Flow SAFs for Weather on I-40

	Medium Rain	Heavy Rain	Light Snow	Light–Medium Snow	Low Visibility	Nonsevere Weather
CAF	0.91	0.84	0.95	0.90	0.90	1.00
SAF	0.93	0.92	0.87	0.86	0.94	1.00

Step 7. Applying the HCM2010 Freeway Facility Analysis Method

The HCM2010 freeway facility analysis method was applied to each of the 2,058 scenarios with capacity and speed–flow curve adjustments appropriate for each scenario.

The standard HCM freeway speed–flow curves are not appropriate when modeling incidents and weather. Therefore, as described in HCM2010, Chapter 37, a modified version of Equation 25-1 from Chapter 25 (Freeway Facilities: Supplemental) is used in combination with the combined capacity adjustment factors (CAFs) and speed adjustment factors (SAFs) to predict basic freeway segment performance under incident and severe-weather scenarios, as shown by Equation I.4:

$$S = (\text{FFS} \times \text{SAF}) + \left[1 - e^{\ln\left((\text{FFS}\times\text{SAF}) + 1 - \frac{C \times \text{CAF}}{45}\right) * \frac{v_p}{C \times \text{CAF}}}\right] \tag{I.4}$$

where

S = segment speed, mph;
FFS = segment free-flow speed, mph;
SAF = segment SAF;
C = original segment capacity, passenger cars per hour per lane (pcphpl); and
v_p = segment flow rate, pcphpl.

CAFs and free-flow SAFs for weather are selected for the I-40 facility based on its free-flow speed of 70 mph, as shown in Table I.13.

The CAFs for segments with incidents on I-40 are selected based on the number of lanes in the subject direction for the segment where the incident is located (Table I.14). The free-flow SAF for incidents is set at 1.00. It is important to note that the factors in Table I.14 do not include the effect of the number of closed lanes. In other words, both the number of lanes closed and the resulting capacity per open lane on the segment must be specified by the user.

For scenarios with both incidents and severe weather, the CAFs are multiplied to estimate their combined effect.

CAFs and SAFs are also applied to the merge, diverge, and weaving segments along the facility, as described in HCM2010, Chapter 37, Travel Time Reliability: Supplemental.

Table I.14. CAFs per Open Lane for Incidents on I-40

Initial Lanes	No Incident	Shoulder Closure	One Lane Closed	Two Lanes Closed	Three Lanes Closed
2	1.00	0.81	0.70	N/A	N/A
3	1.00	0.83	0.74	0.51	N/A
4	1.00	0.85	0.77	0.50	0.52

N/A = not applicable, scenario not feasible.

Step 8. Performing Quality Control and Error Checking and Determining Inclusion Thresholds

Quality control and error checking start with the base scenario (seed file) and proceed to the nonincident, nonsevere weather scenarios.

Error Checks of the Seed File

It is difficult to quality control 2,058 scenarios, so it is recommended that the analyst focus on error checking and quality control on the single initial HCM seed file that is used to generate the 2,058 scenarios. The file should be error checked to the analyst's satisfaction to ensure that it accurately represents real-world congestion on the freeway facility under recurring demand conditions with no incidents and under nonsevere weather conditions. The same criteria for error checking should be used as for a conventional HCM analysis, but with the recognition that any error in the seed file will be crucial, because it will be multiplied 2,058 times by the scenario generator.

Error Checks for Nonincident and Nonsevere Weather Scenarios

Once the seed file has been error checked, the next step is to look at the denied entry statistic for each of the scenarios that do not involve severe weather or incidents. The number of vehicles denied entry to the facility (and not stored on one of its entry links or ramps) should be as near zero as possible for nonsevere weather, nonincident conditions. If feasible, the entry links and ramps should be extended in length to ensure that all vehicle delays for these demand-only scenarios are accounted for within the facility or its entry links and ramps.

The number of vehicles queued on the facility (and its entry links and ramps) during the first analysis period should be nearly the same as the number of vehicles queued in the last analysis period. If necessary, the study period should be extended with one or more artificial analysis periods to ensure that there is not a great change in the number of vehicles queued within the facility between the beginning and the end of the study period. Ideally, the number of vehicles queued in the first and last analysis periods should be zero.

Inclusion Thresholds

As mentioned earlier, the procedure can generate several thousand scenarios, many of which may have exceptionally low or exactly zero probability. In addition, some scenarios may be infeasible. The infeasible scenarios are automatically filtered out by the freeway scenario generation procedure. The scenarios with extremely low probability are not expected to be observed in the field in a single year; however, they are included in the predicted TTI distribution (with an inclusion threshold of zero). Their inclusion makes the comparison of the predicted and observed distributions hard to interpret. In addition, these scenarios tend to have exceptionally large TTI values that significantly shift the tail of the cumulative distribution to the right (i.e., toward higher TTI values).

The procedure allows the user to specify an inclusion threshold to only include scenarios with a probability larger than the threshold specified in the analysis. For instance, an inclusion threshold of 1.0% means that only the scenarios with probability larger than 0.01 are considered in the analysis. Figure I.4 presents the TTI cumulative distributions for four inclusion threshold values for the subject facility, as well

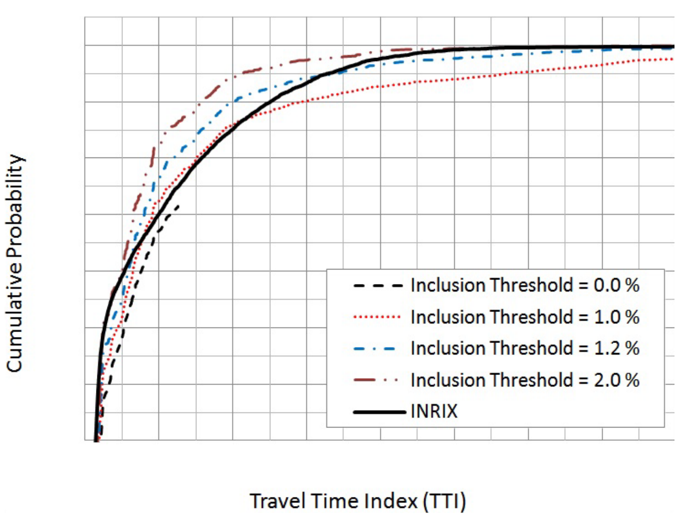


Figure I.4. Travel time distribution results for different inclusion thresholds.

as the observed TTI distribution obtained from the INRIX.com data warehouse. For the subject facility, including all the scenarios with a nonzero probability in the analysis (i.e., an inclusion threshold equal to zero) resulted in a general overestimation in the TTI cumulative distribution. Increasing the threshold to 1.0% brought the TTI distribution much closer to the observed distribution. An inclusion threshold of 1.2% resulted in matching PTI values for the predicted and observed TTI distributions. Inclusion thresholds larger than 1.2% yielded a general underestimation in the TTI distribution.

Increasing the value of the inclusion threshold reduces the number of scenarios and consequently the runtime; however, at the same time it reduces the percentage of the coverage of feasible scenarios. In other words, the larger the value of the inclusion threshold, the higher the number of scenarios excluded from the analysis; thus, fewer numbers of feasible scenarios are covered (see Table I.15).

As Table I.15 shows, the number of scenarios significantly drops as the value of the inclusion threshold increases. By going from an inclusion threshold of 0.00% to 0.01% half of the scenarios were eliminated, the runtime from more than 17 hours to around 6.5 hours was reduced, and the coverage of the distribution was decreased by only 0.29%. This means that more than a thousand of the scenarios contributed to only 0.29% of the TTI distribution.

Step 9. Calculating Performance Measures

The core and supplemental reliability performance measures computed for the example problem are shown in Table I.16. It should be noted that each observation from the I-40 data represents a 15-min mean TTI. For example, the PTI value of 5.34 is interpreted as the TTI associated with the highest fifth percentile analysis period out of all analysis periods covered in the reliability reporting period (in this case, 2,058 × 24 = 49,392 periods). It is critical that when certain TTI parameters are compared with each other that they are computed for identical time periods.

The reliability rating was computed by summing the VMT in all analysis periods with TTI values below 1.33 and dividing the summed results by the sum of VMT in all analysis periods, as shown by Equation I.5:

$$RR = \frac{\sum_{s \in S_{1.33}} VMT_s}{\sum_s VMT_s} \qquad (I.5)$$

where

RR = reliability rating;

$S_{1.33}$ = a set including all analysis periods with TTI values less than 1.33; and

VMT_s = VMT in analysis period s.

Table I.15. Number of Scenarios, Runtime, and Coverage of Feasible Scenarios

Inclusion Threshold	No. of Scenarios	Total Runtime (h:min)	Average Runtime per Scenarios	Coverage of the Distribution (%)
0.00%	2,058	17:18	30.3	100.00
0.01%	1,004	6:31	23.4	99.71
0.10%	496	3:03	22.1	97.46
1.00%	264	1:30	20.5	89.63
1.20%	210	1:05	18.6	85.07
1.30%	174	0:57	19.7	82.55
2.00%	84	0:26	18.6	75.91
3.00%	81	0:24	17.8	67.04
4.00%	4	0:01	15.0	37.32

The PTI was computed by finding the 95th percentile highest analysis period average facility TTI for the subject direction of travel. The 80th percentile TTI was simply the 80th percentile highest TTI (each of which is the average TTI for the analysis period for that scenario).

The semistandard deviation was computed by subtracting one (in essence, the TTI at free-flow speed) from each of the facility average TTIs for each of the analysis periods, squaring each result, weighting each result by its probability, and summing the results. The square root of the summed results was then taken to obtain the semistandard deviation, as shown by Equation I.6:

$$\text{SSD} = \sqrt{\sum_s p_s (\text{TTI}_s - 1)^2} \quad \text{(I.6)}$$

where

SSD = semistandard deviation (unitless);

p_s = probability for analysis period s; and

TTI_s = facility average TTI for analysis period s.

Table I.16. Reliability Performance Measure Results for I-40

Measure	Value
Reliability rating	54.0% (core measure)
Mean TTI	1.97
PTI	5.34
80th percentile TTI	2.03
Semistandard deviation	2.41
Failure/on-time (40 mph)	0.26
Standard deviation	2.21
Misery index	9.39

The failure/on-time index was computed by summing the probability of all analysis periods that have an average speed less than 40 mph, as shown by Equation I.7:

$$\text{FOTI} = \sum_{s \in S_{40}} p_s \quad \text{(I.7)}$$

where FOTI is the failure/on-time ratio, and S_{40} is a set including all analysis periods with average speeds less than 40 mph.

The standard deviation was computed by subtracting the average analysis period TTI (over the reliability reporting period) from each of the facility average TTIs for each of the analysis periods, squaring each of the results, weighting each result by its probability, and summing the results. The square root of the summed results was then taken to obtain the standard deviation, as shown by Equation I.8:

$$\text{SD} = \sqrt{\sum_s p_s \left(\text{TTI}_s - \overline{\text{TTI}}\right)^2} \quad \text{(I.8)}$$

where SD is standard deviation, and $\overline{\text{TTI}}$ is the average analysis period TTI over the reliability reporting period.

The misery index was computed by averaging the highest 5% of travel times divided by the free-flow travel time, or in other words, by averaging the highest 5% TTIs, as shown by Equation I.9:

$$\text{MI} = \frac{\sum_{s \in T_5} p_s \text{TTI}_s}{\sum_{s \in T_5} P_s} \quad \text{(I.9)}$$

where MI is misery index, and T_5 is a set including the highest 5% TTIs.

Table I.17. Evaluation of TTI and PTI Results for I-40

Statistic	I-40 Reliability	Agency Threshold of Acceptability	Conclusion
Mean TTI	1.97	<1.93	Marginally unsatisfactory
PTI	5.34	<3.55	Unsatisfactory

Table I.18. Evaluation of Policy TTI and PTI Results for I-40

Statistic	I-40 Reliability at			Agency Threshold of Acceptability	Conclusion
	70 mph	40 mph	25 mph		
Policy index	1.97	1.13	0.68	>1.00	Unsatisfactory

Step 10. Interpreting Results

This step compares the reliability results with the agency's established thresholds of acceptability and the diagnoses of the major contributors to unreliable travel times on I-40. During the scoping process for this example, the agency selected the mean TTI and the PTI as its reliability performance measures for this study. The calculated TTI and the PTI were compared with the thresholds of acceptable performance established at the start of this example problem. Both statistics fell above the 90th percentile among freeways in the weekday a.m. peak period in the SHRP 2 Project L08 data set, and consequently did not meet the agency's threshold of acceptability for reliable performance (see Table I.17).

The agency's congestion management goal is to operate its freeways at better than 40 mph during 50% of the peak periods of the year and better than 25 mph during 95% of the peak periods during the year. The TTI shown in Table I.17 was recomputed for 40 mph and found to be 1.13 (Table I.18). This value is larger than 1.00, which means that the agency has not achieved this congestion management goal for the I-40 freeway. Similarly, the PTI shown in Table I.17 was recomputed for 25 mph and found to be less than or equal to 1.00, meaning that this goal was achieved.

Reference

Transportation Research Board. *Highway Capacity Manual 2010.* TRB of the National Academies, Washington, D.C., 2010.

APPENDIX J

HCM Urban Streets Methodology Enhancements: Saturation Flow Rate Adjustment Factor for Work Zone Presence

Introduction

There is little research documented in the literature on the effect of work zone presence on urban street operation. Most of the research on the effect of work zone presence on operation has been conducted for freeways. Chapter 10 of the *2010 Highway Capacity Manual* (HCM2010) (Transportation Research Board of the National Academies 2010) provides a synthesis of this research. It indicates that work zone presence tends to reduce the capacity of the freeway lanes that remain open during the work zone. A similar effect is likely to be found on urban streets.

An examination of the nationwide impact of work zones on capacity and delay was conducted by Chin et al. (2004). They used data from Rand McNally Construction Information, the Federal Highway Administration's Fiscal Management Information System, and the Highway Performance Monitoring System to obtain work zone location data and highway capacity data. They modeled the work zone effect on freeways by using capacity adjustment factors documented in the HCM. This same approach was extended to the modeling of work zones on urban streets.

The results of the analysis by Chin et al. (2004) are shown in Table J.1. They estimate that work zones on principal arterials cause about 10 million vehicle hours of delay each year. Freeways are likely to experience more than seven times this amount of delay.

The objective of this appendix is to document the research conducted to quantify the effect of work zone presence on signalized intersection operation. The approach taken in this research is to quantify this effect on intersection saturation flow rate. Data were collected at several intersections for this purpose.

This appendix consists of three main sections that follow this introductory section. The next section summarizes the findings from a review of the literature on the topic of urban street work zones. The third section describes the site selection and data collected for the purpose of quantifying the effect of work zone presence on saturation flow rate. The fourth section describes the findings from an analysis of the field data and the recommended saturation flow rate adjustment factors.

Literature Review

A review of the literature on the topic of urban street work zones focused on work zone factors affecting intersection capacity. However, most of the work zone–related publications found in the literature address freeway operations and safety. In some instances of this review, reference is made to this freeway research when the findings may also be applicable to urban streets.

Work Zone Characteristics

Urban street work zones have several characteristics that differentiate them from highway or freeway work zones. These characteristics are summarized in Table J.2. The focus of this summary is the characteristics that are likely to have a negative influence on urban street traffic operation. In most instances, the influence is likely to be more adverse for the urban street than for the freeway or highway.

A typical intersection work zone is shown in Figure J.1. The work area is shown to be in the lower-left corner of the intersection conflict area. Channelizing devices are used on the eastbound and westbound intersection approaches such that only one lane is open on each approach. This technique facilitates safe intersection operation using flagger direction. The signal is set to a red flash operation.

Work Zone Capacity Studies

Hawkins et al. (1992) measured the capacity of one urban street midsignal work zone in Texas. The work zone was on a

Table J.1. Nationwide Effect of Work Zones on Operation

Highway Type	Work Zone Type	Capacity Reduction, Vehicles/Year (thousands)	Delay, Vehicle Hours/Year (thousands)
Urban freeways and expressways	All	1,702,000	730,000
Urban other principal arterials	All	1,329,000	10,000

Note: Based on 1999 data. Urban freeway and expressway use is 544,000 million vehicle miles; urban other principal arterial use is 393,000 million vehicle miles.

four-lane arterial street. It was a short-term work zone that closed one lane and left one lane open for the subject direction of travel. Hawkins et al. measured the flow rate through the work zone during time periods when a queue was continuously present. They estimated a work zone capacity of 760 vehicles per hour per lane (vphpl) for the open lane.

Relative to a typical capacity of 1,800 vphpl for a traffic lane, the value estimated by Hawkins et al. (1992) suggests that the presence of a midsignal work zone reduces capacity by 1,040 vphpl (58%). This magnitude of reduction is significant and perhaps not typical of most urban street work zones. No other published reports could be found to corroborate the findings by Hawkins et al. The HCM2010 recommends 1,600 vphpl for a short-term freeway work zone, which is considerably larger than 760 vphpl.

Elefteriadou et al. (2008) used a simulation model to estimate work zone capacity when the work zone was in the vicinity of a signalized intersection. They developed a set of regression equations that could be used to predict the capacity based on a variety of factors that describe the signal timing, approach geometry, and distance between the work zone and intersection. They used the calibrated models to determine that, for a single-lane closure on a three-lane approach, the capacity would range from 385 to 1,005 vphpl, depending on the factors mentioned. An examination of the model's regression coefficients indicated that the presence of the work zone at the intersection reduces approach capacity by about 218 vphpl.

Factors Affecting Work Zone Operation

Joseph et al. (1988) developed a simulation model for evaluating work zones on signalized arterial streets. Their research revealed that work zone effect on traffic operation was dependent on work zone location (relative to the signalized intersection), signal timing, and the degree to which arrivals were concentrated in platoons.

Elefteriadou et al. (2008) developed a series of equations for predicting intersection approach capacity when a work

Table J.2. Urban Street Work Zone Characteristics

Category	Characteristic	Relative to Highway and Freeway Work Zones, the Urban Street Work Zone Has . . .
Geometry	Midsignal access	More frequent driveway access, which may disrupt platoon progression by vehicles turning into or out of the major-street work zone and introduce significant speed variation on the urban street.
	Cross section	Undivided cross section in some cases, which reduces the lateral separation between opposing vehicles in many work zone configurations.
		Higher likelihood of right-of-way constraint, which may result in narrow traffic lanes and the need for barrier protection for work zone occupants.
		No shoulders, which may limit work zone configuration options that could otherwise minimize work zone impact on capacity.
Traffic characteristics	Pedestrians	More frequent pedestrians, whose accommodation in the work zone can reduce the right-of-way available to serve vehicles in the work zone.
	Left turns	A larger portion of left-turn vehicles, which could cause increased delay if left-turn capacity is reduced by work zone lane restrictions or queue spillback.
Traffic control	Signals	More frequent signalized intersections, whose coordinated operation is often disrupted by work zone presence and whose detectors are often disabled by construction activities.
	Stop or yield control	High-volume turn movements at unsignalized access points that may not have adequate capacity due to work zone–related queue spillback.

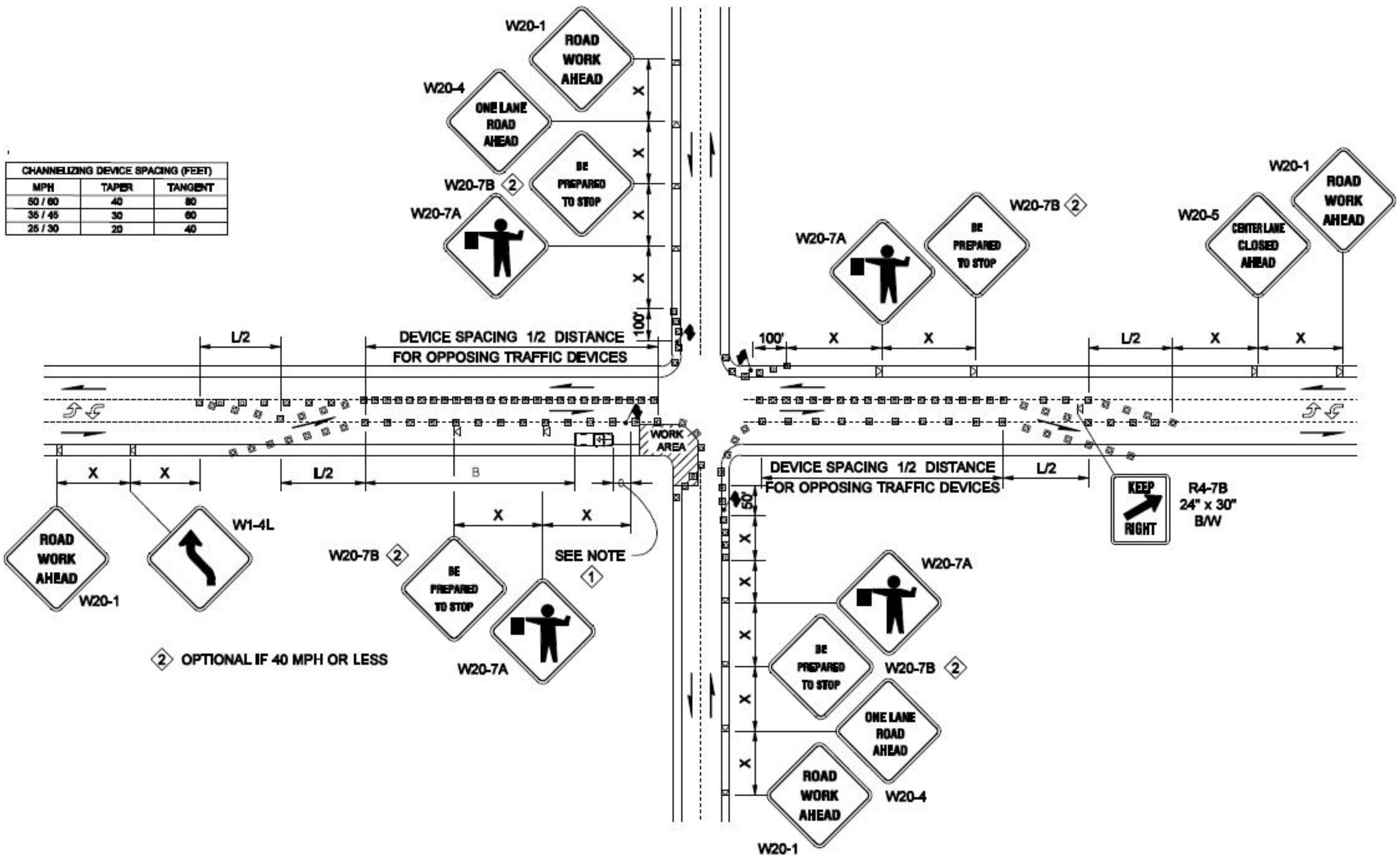


Figure J.1. Typical intersection work zone.

zone was present. Each equation addressed different approach lane configurations at the intersection. The variables in the equations indicated that the capacity of the work zone is a function of the percentage of left-turning vehicles, the distance between the work zone and intersection, and the green–to–cycle length ratio of the lane group.

Hawkins et al. (1992) observed several urban street work zones in Texas. They noted that the following factors had some influence on the operation of an urban street with a midsegment work zone: pedestrian presence, driveway access, barriers that block sight lines, narrow lanes that make it difficult to turn into or out of driveways, and lateral clearance between open lanes and the work zone.

The factors identified in the review of the literature that can affect the capacity of a signalized intersection are summarized in Table J.3. These factors could represent input variables in a model or procedure for predicting intersection capacity. In this regard, the model or procedure would be used to evaluate each intersection approach separately.

Kianfar et al. (2011) conducted a state-of-the-practice survey that included all state departments of transportation in the United States. The survey focused on freeway work zones; however, some of the findings are relevant to the discussion of urban street work zones. One of the questions related to the factors that influence work zone capacity. The factors selected by a majority of the respondents are identified in Column 2 of Table J.3. They include work zone length, number of open lanes, lane width, and heavy-vehicle percentage.

Freeway Work Zone Capacity

Chapter 10 of the HCM2010 provides information for estimating the capacity of freeway work zones. It differentiates between short-term work zones and long-term work zones. Short-term work zones are noted to have standard channelizing devices (e.g., cones, drums) to demarcate the work area and work activities that tend to last a few hours or weeks. Long-term work zones are noted to have portable concrete barriers to demarcate the work area and work activities that tend to last a few months or years.

For short-term work zones, a procedure is provided in the HCM2010 to predict freeway capacity. A base capacity value of 1,600 passenger cars per hour per lane (pcphpl) is recommended. It can be adjusted for the level of work activity, the presence of heavy vehicles, and the presence of ramps.

For long-term work zones, Exhibit 10-14 in the HCM2010 lists default freeway capacity values for selected lane reduction combinations. These values are repeated in Table J.4. Two trends are suggested by these values. One trend is that a

Table J.3. Work Zone–Related Factors that May Affect Intersection Capacity

Category	Factor[a]
Work zone data	Work zone length[a]
	Location of closed lane (outside, middle, or inside; parking)
	Work intensity (presence of equipment and workers)
	Work duration (number of days since work zone installed)
	Police presence
	Time of work activity (daytime, nighttime)
Geometry	Number of open lanes in the work zone[a]
	Approach grade
	Lane width in the work zone[a]
	Lateral clearance to the work zone and to opposing lanes
	Driveway presence
	Provision or closure of turn lanes at intersection
Traffic characteristics	Traffic demand volume
	Heavy-vehicle percentage[a]
	Lane utilization (or lane volume) on intersection approach
	Turn-movement percentages
	Pedestrians at intersection and along street, if sidewalk is closed
Traffic control	Speed limit prior to work zone and speed limit in work zone
	Use of flagger or signal control
	Type of devices used to delineate work zone (cones, barrier, other)
	Effective green duration and cycle length, if signalized

[a]These factors are most frequently considered by practitioners (Kianfar et al. 2011).

long-term work zone has a lower capacity than a short-term work zone.

The second trend in the values shown in Table J.4 relates to the change in capacity with number of lanes. The capacity values shown in the table suggest that capacity per lane is higher for freeways with many normal lanes. It is possible that this trend is confounded with area type (i.e., urban freeways tend to have more lanes and more aggressive drivers than rural freeways).

Table J.4. HCM2010 Default Capacity Values for Long-Term Freeway Work Zones

No. of Lanes		Freeway Capacity[a] (vphpl)
Normal Operation	During Work Zone	
2	1	1,400
2	2	N/A
3	1	1,450
3	2	1,450
3	3	N/A
4	2	1,450
4	3	1,500

[a]Values from Exhibit 10-14 in Chapter 10 of the HCM2010.
N/A = not applicable, data not available.

Freeway work zone capacity has been the subject of several research projects in the past 20 years. Data from the reports associated with several of these projects are listed in Table J.5. Collectively, these data represent a range in number of lanes, proportion of heavy vehicles, and work zone duration.

These data were statistically reexamined to determine if there was an underlying trend between the number of lanes, lanes reduced for work zone, proportion of heavy vehicles, size of lane closure (not shown), work zone duration, and capacity. The regression model described by Equations J.1 to J.4 was used for this evaluation.

$$h_{wz} = b_o \times \mathrm{fh}_{hv} \times \mathrm{fh}_{long} \times \mathrm{fh}_{reduce} \tag{J.1}$$

with

$$\mathrm{fh}_{hv} = 1.0 + p_{hv}\,(b_{hv} - 1.0) \tag{J.2}$$

Table J.5. Reported Capacity Values for Freeway Work Zones

Source	Lanes Open During Normal Operation	Lanes Open During Work Zone	Proportion Heavy Vehicles	Work Zone Duration	Measured Capacity[a] (vphpl)
Benekohal et al. (2003)	2	1	0.294	Long	2,062
	2	1	0.347	Long	1,710
	2	1	0.382	Long	2,088
	2	1	0.061	Long	1,981
	2	1	0.426	Long	1,615
	2	1	0.169	Long	2,167
	2	1	0.189	Long	2,033
	2	1	0.145	Long	2,004
Al-Kaisy and Hall (2002)	3	3	0.0[b]	Long	2,252
	4	4	0.0[b]	Long	1,853
	4	2	0.0[b]	Long	1,989
	4	2	0.0[b]	Long	1,985
Kim et al. (2001)	4	3	0.082	Short	1,612
	4	3	0.081	Short	1,627
	4	3	0.090	Short	1,519
	4	3	0.103	Short	1,790
	4	3	0.080	Short	1,735
	4	3	0.101	Short	1,692
	4	2	0.143	Short	1,290
	4	2	0.085	Short	1,228
	4	2	0.110	Short	1,408
	4	2	0.113	Short	1,265
	4	2	0.046	Short	1,472
	4	2	0.099	Short	1,298
Dixon et al. (1996)	2	1	0.072	Short[c]	1,637
	2	1	0.118	Short[c]	1,644
	2	1	0.045	Short[c]	1,787
	2	1	0.214	Short[c]	1,692
	2	1	0.193	Short[c]	1,440
Jiang (1999)	2	1	0.250	Short[c]	1,500
	2	1	0.120	Short[c]	1,572
	2	1	0.110	Short[c]	1,190
	2	1	0.320	Short[c]	1,308
	2	1	0.310	Short[c]	1,320

(continued on next page)

Table J.5. Reported Capacity Values for Freeway Work Zones (continued)

Source	Lanes Open During Normal Operation	Lanes Open During Work Zone	Proportion Heavy Vehicles	Work Zone Duration	Measured Capacity[a] (vphpl)
Krammes and Lopez (1992)	3	1	0.121	Short	1,304
	3	1	0.129	Short	1,387
	3	1	0.151	Short	1,534
	3	1	0.044	Short	1,665
	3	1	0.105	Short	1,435
	3	1	0.118	Short	1,311
	3	1	0.031	Short	1,470
	3	1	0.133	Short	1,405
	3	1	0.150	Short	1,498
	3	1	0.227	Short	1,502
	3	1	0.174	Short	1,544
	2	1	0.049	Short	1,447
	2	1	0.071	Short	1,539
	2	1	0.032	Short	1,641
	2	1	0.034	Short	1,555
	2	1	0.034	Short	1,478
	2	1	0.028	Short	1,668
	2	1	0.132	Short	1,522
	2	1	0.049	Short	1,521
	2	1	0.034	Short	1,615
	2	1	0.040	Short	1,682
	2	1	0.036	Short	1,661
	4	2	0.167	Short	1,479
	4	2	0.151	Short	1,430
	4	2	0.041	Short	1,860
	4	2	0.085	Short	1,402
	4	2	0.045	Short	1,406
	5	3	0.018	Short	1,681
	5	3	0.021	Short	1,479
	4	3	0.037	Short	1,668
	4	3	0.039	Short	1,471
	4	3	0.037	Short	1,681
	4	3	0.057	Short	1,387

[a] The capacity values were measured, but the measurement technique varies among researchers.
[b] Reported capacity is in terms of equivalent passenger cars per hour per lane.
[c] Duration not stated by author. Short duration assumed from work zone description.

$$fh_{long} = 1.0 + b_{long}\ I_{long} \tag{J.3}$$

$$fh_{reduce} = 1.0 + b_{reduce}\,(n_o - n_{wz}) \tag{J.4}$$

where

h_{wz} = saturation headway when a work zone is present, s/vehicle (s/veh);
fh_{hv} = adjustment factor for heavy vehicles;
fh_{long} = adjustment factor for work zone duration;
fh_{reduce} = adjustment factor for reducing lanes during work zone presence;
p_{hv} = proportion of heavy vehicles;
I_{long} = indicator variable for work zone duration (1.0 if long term, 0.0 if short term);
n_o = number of lanes open during normal operation;
n_{wz} = number of lanes open during work zone presence; and
b_i = regression coefficient i.

The regression model is developed to predict the saturation headway when a work zone is present. This headway is computed by dividing 3,600 by the freeway capacity provided in the far-right column of Table J.5. The regression coefficient b_o represents the equivalent through-vehicle saturation headway for short-term freeway work zones with no lane reduction.

The adjustment factor for heavy vehicles is a variation of Equation 10-8 from the HCM2010. The regression coefficient in Equation J.2 represents the passenger car equivalent for heavy vehicles.

The adjustment factor for reducing lanes was derived following an examination of the trends in Tables J.4 and J.5. Alternative forms of Equation J.4 were explored, but that shown was found to provide the best fit to the data.

The statistics associated with the calibrated model are shown in Table J.6. The coefficient of determination R^2 is .59.

The coefficient b_o suggests that the saturation headway for short-term work zones is 2.0739 s/pc. This value equates to a capacity of 1,736 pcphpl for short-term work zones.

The coefficient b_{hv} has a value of 1.4556. This value is similar in magnitude to the passenger car equivalent for trucks in level terrain of 1.5 that is provided in Exhibit 11-10 of the HCM2010.

The regression coefficient b_{long} represents the effect of work zone duration and demarcation devices. Its value is −0.2371. This value suggests that the saturation headway for long-term freeway work zones is 24% smaller than that for a short-term headway. Alternatively, it suggests that the capacity for the long-term work zone is 31% larger than for a short-term work zone. Al-Kaisy and Hall (2002) rationalize that this increase is likely due to drivers feeling more secure with concrete barriers than plastic barrels, and their greater familiarity with long-term work zones than short-term work zones. However, it is noted that this trend is opposite to that in the capacity values provided in Chapter 10 of the HCM2010.

The regression coefficient b_{reduce} represents the effect of lane reductions through the work zone. The positive value of this coefficient suggests that saturation headway is higher at work zones where there are many lanes closed relative to work zones where there are few lanes closed. This trend may reflect the amount of turbulence in the approaching traffic stream that is forced to merge before reaching the work zone. If there is one lane closed for a work zone, this factor has a value of 1.075. If it is a short-term work zone, then the saturation headway is 2.228 (2.0739 × 1.075), which equates to a capacity of 1,616 pcphpl. This latter value compares favorably with that recommended in Chapter 10 of the HCM2010 for short-term work zones.

Methodological Issues

This subsection describes the appropriate method for estimating saturation flow rate using field data. The accuracy of the

Table J.6. Model Statistical Description: Freeway Saturation Headway During Work Zone

Model Statistic		Value		
R^2		0.59		
Observations n_o		67 sites		
Calibrated Coefficient Values				
Variable	**Inferred Effect**	**Value**	**Std. Dev.**	***t*-statistic**
b_0	Saturation headway for short-term work zones (s/pc)	2.0739	0.0801	25.9
b_{hv}	Passenger car equivalent for heavy vehicles	1.4556	0.1468	9.9
b_{long}	Adjustment factor for long-term work zone	−0.2371	0.0281	−8.5
b_{reduce}	Adjustment factor for lane reduction at work zone	0.0745	0.0261	2.9

saturation flow rate estimate for a specific lane (or lane group) is highly dependent on the method used to aggregate the data that are recorded in each signal cycle. The underlying issue is whether to base the computation of overall saturation flow rate either on individual measurements of average headway per cycle or on individual observations of saturation flow rate per cycle.

The two methods yield estimates of overall saturation flow rate that differ by about 50 vphpl. The reason for the difference in the two methods is due to two factors: (1) the cycle-based statistics (i.e., average headway per cycle and average saturation flow rate per cycle) have a random component, and (2) one statistic is the reciprocal of the other. From a mathematical standpoint, a randomly distributed variable that is converted by reciprocal and averaged will not equal the reciprocal of the average value of the randomly distributed variable.

The appropriate averaging method is the one that yields an unbiased estimate of cycle capacity. Bonneson et al. (2005) demonstrated that average saturation flow rate is accurately computed from individual measurements of average headway per cycle.

Site Selection and Data Collection

This section describes the criteria used to select study sites and the plan established for collecting the data needed to quantify the effect of work zone presence on saturation flow rate. Intersections in several states were considered for inclusion in the database assembled for this project.

For this research, a study site is defined as one intersection approach. At each site, data were collected for the through-lane group. This lane group includes any combination of exclusive through lanes and shared through and right-turn lanes.

The study design is described as an observational during–after study. Data were collected at each study site when the work zone was present, and then again after the work zone was removed. The study was observational because the local transportation agencies selected the intersections requiring maintenance or reconstruction.

The next part of this section describes the site selection criteria and the process used to select the study sites. The third part of this section describes the data collection plan. This plan describes the data to be collected, data collection methods, study duration, and sample size. The last part of this section describes the data reduction procedures.

Site Selection

The selection of suitable study sites was based on a range of criteria. The criteria used were based on the annual average daily traffic (AADT), work zone end date, work zone duration, and number of lanes closed for the work zone. The volume criterion was established as a minimum AADT of 3,550 vehicles per day per lane. This volume was used to maximize the potential for acquiring the desired minimum sample size during a study during one peak traffic period. The work zone end-date criterion was used to ensure that the work zone would be removed in a timely manner, such that the after study could be completed within the time schedule of the research project. The other two criteria were used to guide site selection such that a range of values for each criterion were represented in the database.

In addition to these criteria, the following desirable site characteristics were established to guide the selection process:

- A left-turn bay must be present on any approaches where left-turn movements occur;
- Approaches should not have sharp curves or other unusual horizontal or vertical geometry;
- Approaches should have a grade in the range of –0.5% to +0.5%; and
- Approaches should not experience queue spillback during the study period.

It was determined that a minimum of eight study sites would need to be in the database to collectively represent the desired combinations of work zone duration and number of lanes.

Table J.7 lists the study sites selected for field data collection. The sites represent three states. The approach width in Column 6 describes the total width of open lanes when the work zone is present. It includes the width of the left-turn, through, and right-turn lanes and describes the lateral distance between the work zone channelizing devices (and curb, if the devices are only on one side of the approach).

As stated previously, a long-term work zone typically includes portable concrete barriers to demarcate the work area and work activities that tend to last a few months or years. Only Site 3 had these characteristics. Sites 5 and 7 had the characteristics of a short-term work zone. The other sites had a combination of the characteristics of both categories.

Data Collection

For a given site, the data were collected using two camcorders. The location of these camcorders is shown in Figure J.2.

One camera was mounted on a pole just behind the curb and facing the intersection. This camera was used to determine whether there were at least 10 vehicles in queue at the start of green and the time that the signal indication changed. This camera was positioned such that its field of view included (1) at least one controlling signal head for the subject through and right-turn movements and (2) a view of each traffic lane serving the subject movements (up to three lanes). At most

Table J.7. Study Sites by Location

State	Site No.	Intersection	No. of Left and Through Lanes		Approach Width During Work Zone (ft)	Work Zone Duration (days)	Traffic Control Devices Demarcating Work Area
			After Work Zone	During Work Zone			
Arizona	1	E. Valencia Road and S. Alvernon Way	5	2	26.6	120	Cone
Texas	2	Kirby Drive and US-59 westbound Frontage Road	3	2	31.0	290	Drum
Florida	3	Brickell Avenue and S.E. 13th Street	3	2	20.0	290	Concrete barrier
	4	Sample Road and N.W. 54th Avenue	5	4	50.0	150	Drum
Texas	5	W. Holcombe Blvd. and Buffalo Speedway	4	2	22.0	2	Drum
Arizona	6	N. Sabino Canyon Road and E. Tanque Verde Road	5	4	48.0	100	Cone
	7	E. Thomas Road and N. 46th Street	4	2	22.3	10	Cone
	8	Cave Creek Road and Greenway Pkwy.	4	2	23.9	100	Cone

sites, this camera was located in the range of 300 to 500 ft upstream of the stop line.

The second camera was mounted on a pole just behind the curb at the stop line and facing in a direction perpendicular to the flow of traffic on the subject approach. This camera was used to determine the time that the front axle of each queued vehicle crossed the stop line. The clock in each videotape recorder was synchronized to a master clock at the start of each study.

The cameras recorded traffic events for a minimum of 4 h during each study (i.e., 4 h during work zone operation and 4 h after the work zone was removed). The objective for each site was to record a minimum of 270 vehicles that were in saturation flow (i.e., in Queue Position 5 and higher) during each study. At all sites, the cameras were maintained for longer periods of time to maximize the number of headway observations. Resource constraints limited the camera deployments to a maximum of 40 h of recorded traffic operation at any given site.

The technicians recorded for each site the data described in the following list:

- Street names;
- Approach lane assignments;
- Approach lane width;
- Left-turn bay length and lanes;
- Right-turn bay length and lanes;
- Posted speed limit;
- Bus stop location, if present;
- On-street parking, if present;
- Driveway location, if present;
- Adjacent land use (e.g., office, commercial, residential, industrial), if present; and
- Camera location (i.e., distance from stop line).

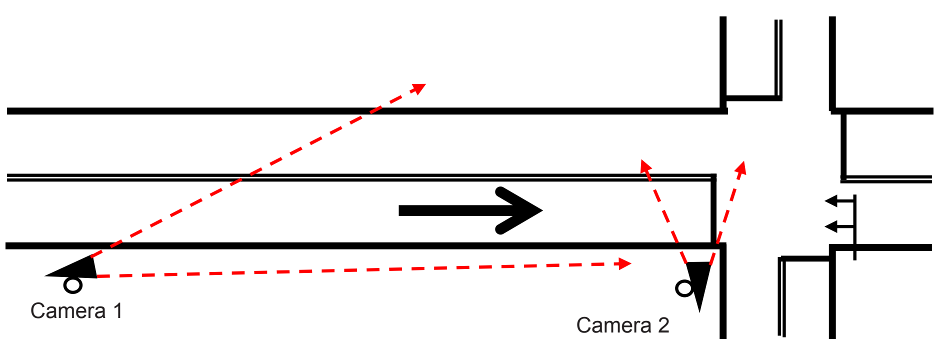


Figure J.2. Typical camera location on the intersection approach.

In addition to the data in the previous list, the data described in the following list were collected at each site that had a work zone present:

- Weather condition;
- Distance from start of work zone to stop line;
- Distance from end of work zone to stop line;
- Type of traffic control devices used to demarcate the work area;
- Number of open lanes on the study approach; and
- Number of work zone–related workers present.

For each work zone study site, the technicians obtained (when available) a copy of the traffic control plan for the work zone and the date the work zone was installed.

Data were collected in the winter of 2011 and spring of 2012 during time periods that are reflective of typical peak traffic periods at each study site. These periods typically occurred during midweek days (i.e., Tuesday, Wednesday, and Thursday) in the morning, afternoon, and evening peak periods. Data were not collected during holidays, periods of inclement weather, or incidents.

Data Reduction

After each study, the videotape recordings were replayed in the office. Saturation headway data were extracted from these recordings using the technique described in Section 6 of Chapter 31 of the HCM2010. The data extracted for each signal cycle included the following items:

- Time of start of green;
- Time of end of green;
- Discharge time of the first queued vehicle;
- Discharge time of the fourth queued vehicle;
- Discharge time of the eighth queued vehicle;
- Discharge time of the last queued vehicle (only used in special circumstances);
- Number of heavy vehicles in Queue Positions 1 through 4;
- Number of heavy vehicles in Queue Positions 1 through 8;
- Number of heavy vehicles in queue (only used in special circumstances);
- For shared-lane groups, number of right-turn vehicles in Queue Positions 1 through 4;
- For shared-lane groups, number of right-turn vehicles in Queue Positions 1 through 8;
- For shared-lane groups, number of right-turn vehicles in queue (only used in special circumstances);
- Number of queued vehicles at the start of green; and
- Number of vehicles served during the cycle.

In those situations when volumes were unexpectedly light, it was necessary to record the times for the fourth and last queued vehicles. In this situation, the last queued vehicle was in Queue Position 6 or 7.

Data Analysis and Findings

This section describes the development and evaluation of several saturation flow adjustment factors that explain the effect of work zone presence on saturation flow rate. The effect of each factor on saturation flow rate, as reported in the literature, was described in a previous section. This section concludes with a description of the equations used to estimate the saturation flow rate for a signalized movement when a work zone is present.

Database Summary

This subsection summarizes the data collected at eight signalized intersection approaches in three states. Each approach represents one field study site. Study site traffic characteristics are summarized in Table J.8. One observation represents the average of the saturation headways measured during one signal cycle. The data represent measurements for 3,429 vehicles in saturation flow during the after study and for 3,772 vehicles during the work zone present study.

Data were not collected at Site 3 for the after work zone condition. This outcome was unexpected because the work period was scheduled for completion during the data collection phase of this project. However, the operating agency extended the work period for unknown reasons such that it was not possible to collect the after data in a time frame that would be useful to the project.

The after and during saturation headway for each site can be compared to assess the effect of work zone presence. For six of the sites, the saturation headway during the work zone was found to be higher than that of the after condition. In contrast, Sites 2 and 4 were found to have slightly larger saturation headways during the after condition than when the work zone was present. It is not clear from the data why this result occurred, but it is contrary to expectation.

Site 3 is the only site considered to have a long-term work zone. Its saturation headway when the work zone was present was 2.62 s/veh. This value corresponds to a saturation flow rate of 1,374 vphpl. This saturation headway is larger than that of any other site. This trend suggests that intersections with long-term work zones have lower capacity than those with short-term work zones. This trend is consistent with that described in Chapter 10 of the HCM2010, but it is contrary to that found in the analysis of the freeway data in Table J.5. This finding suggests that characterizing work zone characterizations as short-term or long-term is insufficient

Table J.8. Study Site Traffic Characteristics

Site No.	After Work Zone				During Work Zone			
	n_o	h_o (s/veh)	p_{hv}	p_{rt}	n_o	h_o (s/veh)	p_{hv}	p_{rt}
1	98	2.20	0.023	0.000	98	2.43	0.105	0.084
2	199	2.18	0.008	0.000	53	2.16	0.009	0.000
3	N/A	N/A	N/A	N/A	75	2.62	0.020	0.147
4	219	2.17	0.021	0.000	257	2.03	0.019	0.000
5	139	2.24	0.045	0.169	162	2.59	0.014	0.134
6	143	1.98	0.005	0.000	141	2.04	0.048	0.009
7	24	2.40	0.021	0.010	84	2.53	0.036	0.033
8	132	2.29	0.030	0.078	73	2.46	0.031	0.038
Summary	954	2.18	0.021	0.036	943	2.30	0.033	0.051

Note: n_o = number of observations of average saturation headway per cycle; h_o = average saturation headway; p_{hv} = proportion of heavy vehicles; p_{rt} = proportion right-turn vehicles in shared through and right-turn lane; N/A = not applicable, data not available.

to describe systematic variation in work zone saturation flow rate or capacity.

The average values in the last row can be used to quantify the approximate effect of work zone presence on saturation flow rate. Specifically, the ratio of the two saturation headways indicates that work zone presence decreases saturation flow rate by about 5.2% ($100 \times [1.0 - 2.18/2.30]$).

Model Development

This subsection describes the development of a regression model that was used to estimate the adjusted saturation headway for each site. The adjusted saturation headway is defined as the saturation headway for an equivalent through-car traffic steam served in a 12-ft traffic lane. This adjusted value is estimated separately for the after and during periods by using Equations J.5 to J.8:

$$T_s = b_{site} \times n_s \times fh_{hv} \times fh_{rt} \times fh_w \tag{J.5}$$

with

$$b_{site} = b_1 + b_2\ I_2 + b_3\ I_3 + b_4\ I_4 + b_5\ I_5 + b_6\ I_6 + b_7\ I_7 + b_8\ I_8 \tag{J.6}$$

$$fh_{hv} = 1.0 + p_{hv}(b_{hv} - 1.0) \tag{J.7}$$

$$fh_{rt} = 1.0 + p_{rt}(b_{rt} - 1.0) \tag{J.8}$$

where

T_s = saturated discharge time, s;
n_s = number of queued vehicles represented in saturated discharge time;
fh_{hv} = adjustment factor for heavy vehicles;
fh_{rt} = adjustment factor for right-turn vehicles;
fh_w = adjustment factor for lane width (1/0.96 if $l_w < 10.0$ ft; 1/1.04 if $l_w > 12.9$ ft; 1.0 otherwise);
l_w = average lane width, ft;
I_i = indicator variable for site i (1.0 if site i, 0.0 otherwise);
p_{hv} = proportion of heavy vehicles;
p_{rt} = proportion of right-turn vehicles;
b_i = regression coefficient for site i ($i = 1, 2, \ldots, 8$);
b_{hv} = regression coefficient for the effect of heavy vehicles; and
b_{rt} = regression coefficient for the effect of right-turn vehicles.

The model described by Equations J.5 to J.8 estimates the adjusted saturated headway by using a method that is consistent with the technique described in Chapter 31 of the HCM2010. The regression coefficients b_i quantify the adjusted saturation headway for each site. The adjusted saturation flow rate is computed post hoc by dividing the adjusted saturation headway into 3,600.

The regression coefficient b_1 defines the adjusted saturation headway of Site 1. The regression coefficient b_2 describes the incremental change in the saturation headway for Site 2 relative to Site 1. Thus, the adjusted saturation headway for Site 2 is computed as the sum of b_1 and b_2. A similar approach is used to determine the adjusted saturation headway for Sites 3 to 8.

The adjustment factor for lane width is based on the saturation flow rate adjustment factor in Chapter 18 of the HCM2010. The reciprocal of the values cited in the HCM are used as headway adjustment factors in Equation J.5.

The regression coefficient b_{hv} represents the passenger car equivalent for heavy vehicles. Similarly, the regression coefficient b_{rt} represents the through-vehicle equivalent for right-turning vehicles. Equivalency factors are cited in Chapter 18 of the HCM2010 for heavy vehicles and right-turn vehicles as 2.0 and 1.18, respectively. These values can vary widely on a cycle-by-cycle basis depending on truck and turn vehicle presence. For this reason, it is appropriate to quantify representative values for these factors as part of the regression analysis.

Unlike the heavy-vehicle and right-turn adjustment factors, the lane width adjustment factor is not derived from the collected data. Rather, the lane width adjustment factor is obtained from the HCM. The reason for this approach is that lane width varies on a site-by-site basis and not on a cycle-by-cycle basis. It is rationalized that the HCM factor values represent the best-estimate lane width effect given that there are only eight sites represented in the database, and that lane width does not vary widely among these sites.

The statistics associated with the calibrated model using after work zone data are shown in Table J.9. The coefficient of determination R^2 is .61.

The coefficient b_{hv} has a value of 1.5115. This value is slightly smaller in magnitude than the passenger car equivalent for heavy vehicles provided in the HCM. It suggests that there is either a larger proportion of small trucks in the observed traffic streams, or that heavy-vehicle performance has improved since the HCM value was quantified.

The coefficient b_{rt} has a value of 1.2076. This value is similar in magnitude to the through-vehicle equivalent for right-turn vehicles provided in the HCM.

The statistics associated with the calibrated model using during work zone data are shown in Table J.10. The coefficient of determination R^2 is .29. This coefficient is about one-half as large as that shown in Table J.9. This trend suggests that there is more random variability in the headways measured at the sites when a work zone is present. This trend is plausible given the added uncertainty in a work zone driving environment.

The adjusted saturation headway for each site was computed using the data in the two previous tables. These headway estimates are shown in Table J.11. The two adjusted values for Site 1 were obtained directly from Tables J.9 and J.10. The value for Site 2 during the after condition was computed as 2.1768 (2.0230 + 0.1538). The values for the other sites and conditions were computed in a similar manner.

The far-right column in Table J.11 compares the two rates using the ratio of after headway divided by during headway. A ratio that is less than 1.0 indicates that the work zone–related saturation headway is larger than the saturation headway for the same movement when there is no work zone.

The ratios in Table J.11 represent an estimate of the average saturation flow rate adjustment factor for work zones. The overall average of 0.90 shown in the last row suggests that work zone presence decreases saturation flow rate by about 10%. This value is larger than that found when examining the unadjusted values shown in Table J.8.

The saturation headways listed in Table J.11 were examined to determine if there was a plausible systematic variation that could be related to the work zone characteristics. A regression analysis was used for this examination.

The adjusted saturation headway for the after condition for Site 3 was not computed because the data were not collected. The overall average value of 2.0903 s/pc was substituted for this missing value for the regression analysis.

For the analysis, the after headway values were compared with the during headway values on a site-by-site basis. This

Table J.9. Model Statistical Description: Saturation Headway After Work Zone

Model Statistic		Value		
R^2		0.61		
Observations n_o		954 cycles (3,429 vehicles)		
Calibrated Coefficient Values				
Variable	**Inferred Effect**	**Value**	**SD**	***t*-statistic**
b_1	Saturation headway of Site 1, s/veh	2.0230	0.0536	37.7
b_{hv}	Passenger car equivalent for heavy vehicles	1.5115	0.0889	17.0
b_{rt}	Through-vehicle equivalent for right-turning vehicles	1.2076	0.0531	22.7
b_2	Incremental saturation headway of Site 2, s/veh	0.1538	0.0599	2.6
b_4	Incremental saturation headway of Site 4, s/veh	0.1285	0.0592	2.2
b_5	Incremental saturation headway of Site 5, s/veh	0.0893	0.0650	1.4
b_6	Incremental saturation headway of Site 6, s/veh	–0.0448	0.0622	–0.7
b_7	Incremental saturation headway of Site 7, s/veh	0.1160	0.1673	0.7
b_8	Incremental saturation headway of Site 8, s/veh	0.0285	0.0705	0.4

Table J.10. Model Statistical Description: Saturation Headway During Work Zone

Model Statistic		Value		
R^2		0.29		
Observations n_o		934 cycles (3,736 vehicles)		
Calibrated Coefficient Values				
Variable	**Inferred Effect**	**Value**	**SD**	***t*-statistic**
b_1	Saturation headway of Site 1, s/veh	2.4248	0.0455	53.3
b_{hv}	Passenger car equivalent for heavy vehicles	1.2744	0.0599	21.3
b_{rt}	Through-vehicle equivalent for right-turning vehicles	1.1641	0.0376	31.0
b_2	Incremental saturation headway of Site 2, s/veh	–0.2727	0.0722	–3.8
b_3	Incremental saturation headway of Site 3, s/veh	0.1222	0.0639	1.9
b_4	Incremental saturation headway of Site 4, s/veh	–0.4037	0.0515	–7.8
b_5	Incremental saturation headway of Site 5, s/veh	0.0946	0.0545	1.7
b_6	Incremental saturation headway of Site 6, s/veh	–0.4153	0.0554	–7.5
b_7	Incremental saturation headway of Site 7, s/veh	0.0736	0.0640	1.2
b_8	Incremental saturation headway of Site 8, s/veh	–0.0018	0.0643	0.0

approach was used to control for other, unmeasured differences among sites. The work zone characteristics that were considered during model development included number of lanes after the work zone was removed, number of lanes when the work zone was present, approach width, work zone duration, and traffic control devices used to demarcate the work area. The values for each characteristic are shown in Table J.7.

Equations J.9 to J.11 were used to model the effect of various work zone characteristics on the adjusted saturation headway:

$$h_{wz} = b_{wz} \times h_o \times \text{fh}_{wid} \times \text{fh}_{reduce} \qquad \text{(J.9)}$$

Table J.11. Estimated Adjusted Saturation Headway

Site No.	Adjusted Saturation Headway After Work Zone (s/pc)	Adjusted Saturation Headway During Work Zone (s/pc)	Ratio (after/during)
1	2.0230	2.4248	0.83
2	2.1768	2.1522	1.01
3	N/A	2.5470	N/A
4	2.1515	2.0211	1.06
5	2.1123	2.5195	0.84
6	1.9782	2.0096	0.98
7	2.1390	2.4984	0.86
8	2.0515	2.4229	0.85
Average	2.0903	2.3244	0.90

N/A = not applicable, data not available.

with

$$\text{fh}_{wid} = 1.0 + b_{wid}(a_w - 12) \qquad \text{(J.10)}$$

$$\text{fh}_{reduce} = 1.0 + b_{reduce}(n_o - n_{wz}) \qquad \text{(J.11)}$$

where

h_{wz} = adjusted saturation headway during work zone, s/pc;
h_o = adjusted saturation headway after work zone, s/pc;
fh_{wid} = adjustment factor for approach width;
fh_{reduce} = adjustment factor for reducing lanes during work zone presence;
a_w = approach lane width during work zone (total width of all open left-turn, through, and right-turn lanes), ft;
n_o = number of left-turn and through lanes open during normal operation;
n_{wz} = number of left-turn and through lanes open during work zone presence;
b_{wz} = regression coefficient for the effect of work zone presence;
b_{wid} = regression coefficient for the effect of approach lane width; and
b_{reduce} = regression coefficient for the effect of reducing lanes.

The statistics associated with the calibrated model are shown in Table J.12. The coefficient of determination R^2 is .81. The R^2 adjusted for sample size is .73. It is recognized that there are only eight sites in the database and that the model

Table J.12. Model Statistical Description: Saturation Headway Adjustment Factors

Model Statistic		Value		
R^2		0.81		
Adjusted R^2		0.73		
Observations n_o		Eight sites		
Calibrated Coefficient Values				
Variable	**Inferred Effect**	**Value**	**SD**	***t*-statistic**
b_{wz}	Adjustment factor for work zone presence	1.1654	0.0670	17.4
b_{wid}	Adjustment factor for work zone approach width	–0.0057	0.0012	–4.9
b_{reduce}	Adjustment factor for lane reduction at work zone	0.0402	0.0265	1.5

has three regression coefficients. Hence, the coefficient of determination is likely to be larger than would truly be obtained if there were more sites in the database. The adjusted R^2 value accounts for the small sample size to some degree.

The regression coefficient b_{wz} represents the effect of work zone presence on saturation headway. Its value of 1.1654 indicates that saturation headway increases 16.54% when a work zone is present. Alternatively, saturation flow decreases by 14% when a work zone is present.

The regression coefficient b_{wid} represents the effect of approach width on saturation headway. It describes the lateral distance between the work zone channelizing devices (and curb, if the devices are only on one side of the approach). The coefficient value of –0.0057 indicates that headway decreases with increasing approach width. Thus, the adverse effect of work zone presence on performance is lessened if there are many open lanes (or a few wide lanes) on the approach.

The regression coefficient b_{reduce} represents the effect of lane reductions through the work zone. This coefficient is slightly smaller than that found in the regression analysis of the freeway data listed in Table J.5. The positive value of this coefficient suggests that saturation headway is higher at work zones where there are many lanes closed relative to work zones where there are few lanes closed. This trend may reflect the amount of turbulence in the approaching traffic stream that is forced to merge before reaching the work zone.

Figure J.3 compares the predicted headway values from Equation J.9 with those from Column 3 of Table J.11 (i.e., the dependent variable). The trend line shown is not the line of best fit. Rather, it is an $x = y$ line such that each data point would lie on this line if the predicted value equaled the measured value. There are eight data points shown in the figure, one data point for each site. They are shown to vary around the line, with no apparent bias over the range of predicted values.

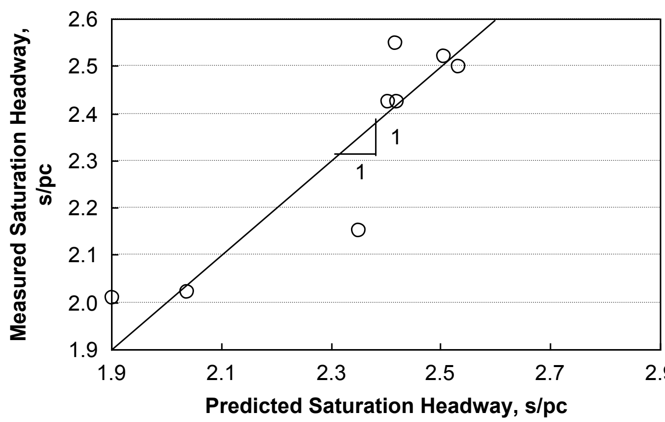


Figure J.3. Comparison of measured and predicted saturation headways.

Saturation Flow Rate Adjustment Factors

This subsection describes a series of equations that can be used to compute a saturation flow rate adjustment factor for work zone presence. This factor would be used with the procedure in Chapter 18 of the HCM2010 to estimate the saturation flow rate of a lane group when there is a work zone on the associated intersection approach.

The saturation flow rate adjustment factor can be computed using Equations J.12 to J.14:

$$f_{wz} = 0.858 \times f_{wid} \times f_{reduce} \quad \le 1.0 \tag{J.12}$$

with

$$f_{wid} = \frac{1.0}{1.0 - 0.0057(a_w - 12)} \tag{J.13}$$

$$f_{reduce} = \frac{1.0}{1.0 + 0.0402(n_o - n_{wz})} \tag{J.14}$$

Table J.13. Illustrative Saturation Flow Rate Adjustment Factor Values

No. of Left and Through Lanes		Approach Width During Work Zone[a] (ft)	Factor for Work Zone Presence	Factor for Approach Width	Factor for Lane Reduction	Combined Factor Value	Predicted Saturation Flow Rate[b] (vphpl)
After Work Zone	During Work Zone						
2	1	11	0.858	0.994	0.961	0.820	1,476
2	2	22	0.858	1.060	1.000	0.910	1,638
3	1	11	0.858	0.994	0.926	0.790	1,421
3	2	22	0.858	1.060	0.961	0.875	1,575
3	3	33	0.858	1.136	1.000	0.975	1,755
4	2	22	0.858	1.060	0.926	0.842	1,516
4	3	33	0.858	1.136	0.961	0.937	1,687
4	4	44	0.858	1.223	1.000	1.000[c]	1,800
5	3	33	0.858	1.136	0.926	0.902	1,624
5	4	44	0.858	1.223	0.961	1.000[c]	1,800
5	5	55	0.858	1.325	1.000	1.000[c]	1,800

[a] Based on an average lane width of 11 ft per lane during work zone presence.
[b] Based on a saturation flow rate of 1,800 vehicles per hour per lane without work zone, after adjustment for other conditions (e.g., grade).
[c] Value rounded down to 1.00.

where

f_{wz} = saturation flow rate adjustment factor for work zone presence;
f_{wid} = saturation flow rate adjustment factor for approach width;
f_{reduce} = saturation flow rate adjustment factor for reducing lanes during work zone presence;
a_w = approach lane width during work zone (total width of all open left-turn, through, and right-turn lanes), ft;
n_o = number of left-turn and through lanes open during normal operation; and
n_{wz} = number of left-turn and through lanes open during work zone presence.

Equation J.12 produces values less than 1.0 for a wide range of conditions. However, when the approach has many lanes open while the work zone is present (or a few wide lanes), then Equation J.12 can mathematically produce a value that exceeds 1.0. In these few instances, a value of 1.0 is recommended as an upper bound on the factor value.

Table J.13 illustrates the value of the factor predicted by Equation J.12 for typical work zone conditions. The values obtained from Equation J.12 are shown in Column 7. The values are shown to range from 0.790 to 1.000. If the saturation flow rate for a given intersection lane group is 1,800 vphpl when no work zone is present, then the saturation flow rate for this lane group when a work zone is present is shown in the far-right column of Table J.13. These saturation flow rate values are consistent with the freeway capacity values shown in Table J.4.

The equations described in this section were calibrated using data for through-lane groups. However, it is suggested that the computed adjustment factor can also be used to estimate the saturation flow rate for left- and right-turn movements from exclusive lanes.

The findings from the literature review were inconclusive regarding the effect of work zone duration (i.e., long term, short term) on saturation flow rate. The data collected for this project were not sufficient in number to shed further light on this issue. It appears that these two designations mask the many underlying factors that truly do have an effect on traffic operation when work zones are present. It is likely that future research on work zone capacity will be more fruitful if researchers abandon the use of descriptors such as long term and short term and instead focus on the individual characteristics of the work zone that may truly be influencing driver behavior.

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Identification and Evaluation of the Cost-Effectiveness of Highway Design Features to Reduce Nonrecurrent Congestion (L07)