Identification and Evaluation of the Cost-Effectiveness of Highway Design Features to Reduce Nonrecurrent Congestion

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The Second
STRATEGIC HIGHWAY RESEARCH PROGRAM

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The Second Strategic Highway Research Program

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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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The research reported in this document was performed by MRIGlobal, supported by HDR Engineering, Inc. Ingrid Potts, MRIGlobal, was the principal investigator. The other authors of this report include Douglas Harwood, Jessica Hutton, Chris Fees, Karin Bauer, and Lindsay Lucas of MRIGlobal and Christopher Kinzel and Robert Frazier of HDR Engineering, Inc. The authors acknowledge the contributions to this research from John O'Laughlin of Delcan and Rachel Powers of Vector Communications.

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FOREWORD

Ralph Hessian, P.Eng., FITE, *SHRP 2 Special Consultant, Capacity and Reliability*

This research report presents the findings on the identification and evaluation of the use of highway geometric design features on freeways to reduce nonrecurrent congestion and improve travel time reliability. General guidance is provided on the range of design elements that could be used by transportation agencies to improve travel time reliability and reduce nonrecurrent congestion, analysis procedures and models to measure their operational and safety effectiveness, as well as a life-cycle benefit–cost method to support decision making on the possible use of individual treatments to address actual nonrecurring traffic conditions. For the safety effectiveness analysis, a new relationship between safety and congestion was explored and a mathematical model developed to quantify crash frequency at various levels of service.

Traffic congestion continues to grow on the nation's highways, which is increasing the concerns of transportation agencies, the business community, and the general public. Congestion includes recurring and nonrecurring components. Recurring congestion reflects routine day-to-day delays during specific time periods where traffic demand exceeds available roadway capacity. Road users have come to expect these daily traffic patterns and adjust their travel plans accordingly to achieve timely arrivals. Nonrecurring congestion that causes unexpected extra delays results from random incidents, such as crashes, weather, and work zones. Road users are frustrated by these unexpected delays that can make for unreliable arrival times at their destinations. The SHRP 2 Reliability research objective focuses on reducing nonrecurring congestion through incident reduction, management, response, and mitigation. Achieving this objective will improve travel time reliability for both people and freight.

Highway geometric design, which involves the provision of physical elements and their dimensions, plays a major contributory role in the traffic operations conditions and safety performance results on highway facilities. Current design standards and guidance manuals do not address the use and effectiveness of design elements as an explicit countermeasure to prevent or mitigate the negative effects of nonrecurring factors and events that happen on a regular basis causing unreliable travel for highway users. This research seeks to better understand how different geometric design elements can contribute to more-reliable travel times and develop procedures that will allow agencies and professional practitioners to evaluate the cost-effectiveness of alternative design elements as a potential solution to specific nonrecurring conditions.

The research team established a list of the physical design elements that could be used to influence nonrecurring congestion. Guidance based on published information and interviews with select transportation agencies is provided for each element. This guidance provides information on advantages and disadvantages, typical applications, expected benefits, factors to consider when selecting the treatment, design criteria, operational and safety effectiveness, and costs. In addition, analysis procedures and models were developed to provide the quantitative measurement of the operational effectiveness, safety effectiveness, and life-cycle cost–benefit results to estimate the potential impact of any candidate design element to improve travel time reliability and reduce nonrecurring congestion.

This research has generated two companion products that allow transportation agencies and professionals to apply these research findings effectively in daily practice. These products are the *Design Guide for Addressing Nonrecurrent Congestion*, which is a catalogue of the design elements and their associated use information, and the *Analysis Tool for Design Treatments to Address Nonrecurring Congestion*, which is a tool to execute the various analysis procedures and models to measure the effectiveness of a design element on travel time reliability.

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Executive Summary

The Reliability area of the second Strategic Highway Research Program (SHRP 2) has focused on the need to improve travel time reliability on freeways and major arterials. SHRP 2 Project L07, Identification and Evaluation of the Cost-Effectiveness of Highway Design Features to Reduce Nonrecurrent Congestion, focused specifically on design treatments that can be used to improve travel time reliability. The objectives of this research were to (1) identify the full range of possible roadway design features used by transportation agencies to improve travel time reliability and reduce delays from key causes of nonrecurrent congestion, (2) assess their costs and operational and safety effectiveness, and (3) provide recommendations for their use and eventual incorporation into appropriate design guides. The research focused on geometric design treatments that can be used to reduce delays due to nonrecurrent congestion.

Highway agencies tend to address recurrent congestion issues with infrastructure treatments and nonrecurrent congestion with intelligent transportation system treatments. That is, daily demand peaks that cause peak hour congestion are often treated by adding base capacity. Congestion caused by incidents, special events, work zones, weather, demand surges, and other infrequent and unpredictable events are typically addressed by providing travelers with real-time information from traffic management centers. These centers monitor freeways and post information about travel time, lane blockages, and alternate routes to drivers in real time via radio, websites, and message boards. Geometric design treatments that address base capacity issues have been investigated and evaluated thoroughly in the literature. More recently, operations-based treatments such as real-time traveler information and motorist-assist patrols have been evaluated for their effectiveness at alleviating nonrecurrent congestion. However, there is a gap in the literature regarding the use of geometric design treatments to help reduce nonrecurrent congestion: Project L07 research helps to fill that gap.

Through interviews with highway agencies, the research team identified instances of agencies using design elements to help manage nonrecurrent congestion; however, in most cases these design treatments had not been designed for this purpose. Instead, treatments designed to manage recurrent congestion were applied to nonrecurrent congestion events, frequently in an ad hoc fashion. When major incidents occurred, agencies used whatever tools were at their disposal to minimize the disruption to traffic. Although these tools were often not design elements put in place specifically to address nonrecurrent congestion, the operational concepts behind them helped the research team develop a list of design treatments that could be implemented to help achieve the same goals more effectively. These goals involved minimizing the time that stalled or crash-involved vehicles blocked lanes, adding temporary capacity to alleviate congestion (e.g., by allowing shoulder driving), providing opportunities for vehicles to escape a queue and find a new route (e.g., by using median gates), reducing both primary incidents (such as truck ramps) and secondary incidents (such as extra-height median walls that prevent rubbernecking behavior), and minimizing the negative impact of weather on the road surface (e.g., by using anti-icing systems).

Operational, safety, and benefit–cost analyses of the design treatments were conducted to achieve the research objectives. The traffic operational analysis methodology developed in this research built on work completed in SHRP 2 Project L03, Analytical Procedures for Determining the Impacts of Reliability Mitigation Strategies, which preceded this research effort. Project L03 developed models for predicting a travel time index (TTI) at five percentiles (10th, 50th, 80th, 95th, and 99th) along the TTI distribution. The TTI distribution represents the travel time of each trip made across a freeway segment during a long time period (for the present purposes, 1 year) relative to the travel time at free-flow speed. That is, vehicles traveling at free-flow speed have a TTI of 1.0, and vehicles traveling at half the free-flow speed have a TTI of 2.0. A full distribution of TTIs for a segment over the course of 1 year captures the travel time of all the trips made, ranging from trips made under free-flow conditions to those made during extreme congestion. Several measurements of delay and reliability can be made from the TTI distribution. The input variables to the Project L03 TTI models were LHL, a measure of lane hours lost due to incidents and work zones; $R_{0.05''}$, the number of hours during the year that rainfall is greater than or equal to 0.05 in.; and *d/c,* the demand-to-capacity ratio for the roadway segment.

The Project L03 models focused primarily on estimating the TTI distributions during peak periods; however, to evaluate the impact of nonrecurrent congestion design treatments on delay and reliability, the analysis needed to include all 24 h of the day. The L07 research team adapted the Project L03 models for use during 1-h time-slices, so that the TTI distribution could be predicted for each hour of the day. In addition, the research team improved on the models in two important ways. First, the Project L03 models were based on data from cities that did not experience significant snowfall, so the present research incorporated a snowfall variable ($S_{0.01''}$) in addition to the rainfall variable in the models. Second, the Project L03 models were developed for peak hours in large metropolitan areas. This research developed additional models to be used for facilities and hours of the day with lower *d/c* ratios (i.e., less than 0.8).

The TTI models as modified for the L07 research were used to estimate and plot the cumulative TTI distributions for each hour of the day. The shape of the cumulative TTI curve provides a great deal of information about delay and reliability. To measure the impact that a specific design treatment has on reliability, the research team developed a method of measuring the difference between TTI curves for "untreated" and "treated" conditions. To develop the curve for the treated condition, the impact of the design treatment must be described in terms of the four model input variables. In general, most design treatments affect the LHL variable by minimizing the number of incidents that occur, reducing the time that lanes are closed or blocked by traffic incidents or work zones, or providing extra capacity during events that close lanes. Hours of rain or snowfall cannot be affected by design treatments, but their impacts on lane capacity can be affected by design treatments such as snow fences and anti-icing treatments. Some design treatments also affect the *d/c* ratio. Once the impacts on these variables are determined for a given design treatment, the delay reduction and improvement in reliability can be measured by analyzing the difference between the two TTI curves.

For the safety analysis of nonrecurrent congestion treatments, this research explored the relationship between congestion and safety—specifically the relationship between level of service (LOS) and crash frequency—and developed a mathematical model to quantify the increase in crash frequency at all severity levels as LOS worsens. Crash frequency is lowest around LOS B and C, but begins increasing through LOS D, E, and F. This relationship indicates that if improvements can be made to LOS (by decreasing congestion), crash frequency will decrease. Therefore, design treatments that reduce congestion also improve safety.

Many design treatments have direct safety benefits in that they reduce the frequency of primary or secondary incidents on the road, but design treatments that also reduce congestion have an indirect safety benefit that can be estimated by using the safety–congestion relationship.

The third treatment analysis was a benefit–cost evaluation for the various design treatments. To calculate treatment benefits, three main components are considered: delay savings, reliability improvement, and safety improvement. By using the untreated (base condition) TTI curve and the treated (after treatment implementation) TTI curve, a reduction in delay due to treatment implementation can be calculated. This measurement, which is expressed in vehicle hours, can be converted to dollars by assigning a monetary value to travel time. Many agencies have a default value that is typically used to convert delay hours to economic cost in dollars. A change in reliability can also be determined on the basis of the shift in TTI cumulative curves from untreated to treated conditions. In this project, reliability was quantified as the standard deviation of the travel time distribution, converted into units of hours. There is no consensus in the literature on how this measure should be valued in economic terms, but one common method is to use a reliability ratio. A reliability ratio is the ratio of the value of reliability to the value of time. By defining this ratio as a fixed number, the value assigned to reliability is always a multiple of the value of time. Just as the value of time may vary from one user group to the next (such as freight or peak hour commuters), so too can the reliability ratio vary from one group to the next. The research team defined the reliability ratio to be 0.8 for all travelers at all times of day in this research; this value fell within the range of most values presented in the literature.

The results of this research provide a method for incorporating both the economic savings due to delay reduction and the economic savings due to reliability improvement for a design treatment over its life cycle. Design treatments that are commonly used to address recurrent congestion can also be analyzed by using the approach developed in this research, which takes into account not only the delay improvements associated with the treatment, but also the potential improvements to reliability. Taking these benefits into account results in a more accurate valuation of a design treatment's net present benefit and benefit–cost ratio. In addition, agencies considering removing roadway features beneficial to nonrecurrent congestion in order to alleviate recurrent congestion (such as by converting a shoulder to a driving lane) can use the methods presented in this report and the Analysis Tool to calculate the expected increase in nonrecurrent congestion and decrease in reliability that might be expected due to the change and compare this cost to the benefits achieved for recurrent congestion by adding additional capacity.

In addition to the documentation of the research in this final report, the research plan included the development of two key products: a design guide for nonrecurrent congestion treatments and an information dissemination plan. The Design Guide catalogs the design treatments considered in this research, providing planners, designers, operations engineers, and decision makers with a toolbox of possible options for addressing nonrecurrent congestion through design treatments. The Dissemination Plan provides a strategic approach to disseminating the results of the research to practitioners to increase awareness of the benefits of designing for reliable roadways.

Through the course of conducting the traffic operational analysis and applying reliability models to assess the traffic operational effectiveness of design treatments, the research team also developed a spreadsheet-based analysis tool that uses the procedures described in this report to provide users with a benefit–cost ratio for various nonrecurrent congestion design treatments on the basis of information input by the user about the specific freeway segment on which it will be implemented, as well as about how the treatment is expected to be implemented. This analysis tool, which is accompanied by a user guide, represents a third key product in the research.

CHAPTER 1

Introduction

Background

The Reliability area of the second Strategic Highway Research Program (SHRP 2) has focused on the need to improve travel time reliability on freeways and major arterials. The objectives of the present research were to (1) identify the full range of possible roadway design features used by transportation agencies to improve travel time reliability and reduce delays from key causes of nonrecurrent congestion, (2) assess their costs and operational and safety effectiveness, and (3) provide recommendations for their use and eventual incorporation into appropriate design guides. The research focused on geometric design treatments that can be used to reduce delays due to nonrecurrent congestion.

Recurrent Congestion Versus Nonrecurrent Congestion

Congestion and consequent delay to motorists result from both recurrent and nonrecurrent congestion.

Recurrent Congestion

Recurrent congestion is regularly occurring, predictable congestion that is generally experienced on a daily basis. On freeways and major arterials, recurrent congestion is generally caused by traffic demand on a facility nearing or exceeding a facility's capacity, and it is most frequently associated with commuter travel during the morning and evening peak periods. On local roads and at intersections, recurrent congestion can also be caused by daily recurring events such as afternoon school dismissals or shift breaks at large employment sites. Recurrent congestion has traditionally been addressed through the design or redesign of highways, bridges, and intersections on which it has occurred or is expected to occur.

Nonrecurrent Congestion

Nonrecurrent congestion arises from random events that are generally unpredictable to the facility user, vary in degree from day to day and from one incident to the next, and create unreliable travel times that frustrate motorists. Sources of nonrecurrent congestion include the following:

- *Traffic incidents.* Traffic incidents are events that disrupt the normal flow of traffic and often involve a blockage of one or more travel lanes. Incidents include such events as vehicle crashes, disabled vehicles, and debris in the travel lane.
- *Weather.* Reduced visibility and roadway surface friction can affect driver behavior and, as a result, traffic flow. Drivers will usually lower their speeds and increase their headways when poor weather conditions are present.
- *Demand fluctuations.* Demand fluctuation refers to the day-to-day variability in traffic demand that leads to higher traffic volumes on some days than on others. Fluctuating traffic demand volumes also result in variable travel times.
- *Work zones.* Work zones are sections of the roadway, or roadside, on which construction, maintenance, or utility work activities take place. Work zones may involve a reduction in the number or width of travel lanes, lane shifts, lane diversions, reduction or elimination of shoulders, or temporary roadway closures.
- *Special events.* Special events include such occasions as major sporting events, festivals, concerts, and even seasonal shopping. Such events cause the traffic flow in the vicinity of the event to differ radically from typical patterns. Special events may cause surges in traffic demand that overwhelm the system.
- *Traffic control devices.* Intermittent disruption of traffic flow by malfunctioning or poorly timed signals or by railroad grade crossings contributes to congestion and travel time variability.

Nonrecurrent congestion has not traditionally been addressed through highway design. In recent decades, operational solutions such as intelligent transportation systems and incident management techniques have been the chief means of combating nonrecurrent congestion. However, highway designers are more frequently considering infrastructure that directly addresses nonrecurrent congestion and that supports or facilitates operational strategies for addressing nonrecurrent congestion during roadway design and redesign projects.

Nonrecurrent congestion is the cause of unpredictable delay; reliability is the measurement of its effects. As the frequency and severity of nonrecurrent congestion events on a facility increase, the reliability of that facility decreases.

Definition of Reliability and Key Terms

Reliability, which is shorthand for travel time reliability, is an important component of roadway performance and, perhaps more importantly, of motorists' perceptions of roadway performance. Having accurate information about roadway performance significantly improves motorists' perceptions of a trip because such information allows motorists to make decisions that give them more control over their trip. Reliability has not been widely used to describe performance, but increasingly agencies are recognizing its value in assessing their own performance and in communicating performance to the public. Reliability and key terms related to reliability are defined below.

Definition of Reliability

Travel time reliability is a relatively new concept. Although various definitions of reliability have been proposed in the literature, no single definition has been universally accepted among traffic operations researchers and practitioners.

Project L07 adopted the working definition for reliability that was developed by the research team for SHRP 2 Project L03, Analytical Procedures for Determining the Impacts of Reliability Mitigation Strategies (*1*):

> *Reliability:* The level of consistency in travel conditions over time, measured by describing the distribution of travel times that occur over a substantial period of time.

This definition of reliability has two key parts:

- Consistency in travel conditions, which refers to consistency in travel times and is mathematically represented by a statistical distribution of travel times.
- Substantial period of time, which for convenience and practicality has been defined in Projects L03 and L07 as 1 year. A period of 1 year also ensures a substantial enough data set on which to draw conclusions about how a facility generally operates.

The measurement and prediction of reliability are mathematically rigorous. Therefore, several terms and concepts are presented here to set the foundation for analyzing travel time reliability later in this report.

Time-Slice

Because the reliability of a roadway may change throughout the day with changing traffic patterns and changing probability of nonrecurrent congestion events, it is evaluated for specific time-slices. A *time-slice* is a single- or multihour portion of a 24-h day, considered over an entire year (excluding weekends and holidays). For example, "the hour from 6:00 to 7:00 a.m. for every nonholiday weekday between January 1 and December 31 of this year" is a single-hour time-slice. Single-hour time-slices are the simplest to work with because they are consistent with the way in which highway traffic volume data are typically collected and analyzed. One way to think of a single-hour time-slice is as an hour-year.

Multihour time-slices defined and evaluated by Project L03 included the following:

- Peak period: a continuous time period of at least 75 min during which the space–mean speed is less than 45 mph
- Midday: 11:00 a.m. to 2:00 p.m.
- Weekday: all 24 h aggregated

In this research, only a single-hour time-slice was used for evaluation.

Travel Time Index

Although expected and actual travel times for a given highway segment or trip are intuitive measures for most drivers ("it should take me 15 minutes to travel from X to Y, but it actually took me 17 minutes"), they are not necessarily convenient universal measures because analysis segments vary in length. Longer segments naturally require longer times to traverse, and comparison of travel times among segments of varying lengths would not be very meaningful. Thus, a numerical travel time measure exhibiting consistency across facilities of varying lengths is desirable. In reliability research, the travel time index (TTI) has emerged as such a measure. TTI is defined as the ratio of the actual time spent traversing a given distance to the free-flow travel time for that same distance.

TTI can be measured at the scale of individual vehicles. For example, if the free-flow speed of a 2-mi freeway segment is 60 mph (meaning a vehicle could traverse the segment in 2.0 min), and a vehicle traverses the segment in 2.4 min, then the TTI for that vehicle is the ratio of 2.4 to 2.0 min (i.e., TTI = 1.2; note that as a ratio of two quantities measured in consistent units of time, TTI is a unitless index). For reliability analysis, however, it is useful to aggregate TTI to larger scales, rather than at the scale of individual vehicles, such as all vehicles traversing a segment during a time-slice.

At least two other measures, travel speed and travel rate, could be considered as fundamental measures of reliability, as each "normalizes" for both travel time and segment length:

- Travel speed is expressed in the familiar units of miles per hour (the ratio of time to distance).
- Travel rate is essentially the inverse of travel speed, expressed as a ratio of distance to time (e.g., seconds per mile).

However, as a unitless measure, TTI is a preferable standard because it can compare across different facilities regardless of the speed for which they are designed. For example, an average travel speed of 55 mph (travel rate of 65 s/mi) would be quite acceptable on a facility with a free-flow speed of 55 mph (65 s/mi), but it would be less acceptable on a facility with a free-flow speed of 70 mph (51 s/mi). In each case, the analyst would need two numbers to judge the reliability of the facility: the actual and free-flow speeds (or travel rates). In contrast, a single TTI value (a reliable 1.0 in the first case and a less reliable 1.27 in the second) would be sufficient to make this judgment.

Because TTI is defined relative to the free-flow speed of the facility, motorists traveling faster than the free-flow speed have a TTI value less than zero. For the purposes of this research, the 90th percentile speed (corresponding to the 10th percentile TTI) was used as a surrogate for free-flow speed, and any TTI values less than 1.0 were set equal to 1.0.

Scope and Scale of Reliability

Travel time and reliability can generally be considered from two perspectives:

- *Facility based.* At the smallest scale, travel time can be considered over a short, uninterrupted, homogenous highway segment for all vehicles that travel the segment over a time-slice. Such facility-based measures could be extended to a highway corridor and ultimately to an entire metropolitan highway system.
- *Trip based.* As experienced by the individual traveler, trip-based travel times are ultimately what truly matters. For example, an individual commute typically traverses numerous facility types and segment lengths, and the reliability of each contributes to the reliability of the entire commute.

As the most microscopic measure, segment-based travel times can be aggregated to derive any other scale. For example, as described above, an individual trip is composed of a series of segments. And certainly, smaller segments (or at most, corridors) are the scale at which design decisions and investments are made.

Reliability statistics can be disaggregated by travel mode (automobile, truck, transit), travel purpose (freight movement, commute to work, business travel, personal errands, leisure travel), or by both mode and purpose. Such categorizations are especially useful for economic evaluations in which reliability may be valued differently for different trip purposes.

Fundamental Diagram of Reliability

Reliability is described by a distribution of travel times. Graph A in Figure 1.1 illustrates a typical travel time probability distribution function (TT-PDF) for travel times on a freeway segment. Such distributions can have many shapes and are not always unimodal (single-peaked).

Graph B shows the same distribution presented as a travel time cumulative distribution function (TT-CDF). The resulting S-curve shape, with a standardized vertical axis, allows easy visual extraction of cumulative percentiles, including the median (50th percentile).

By incorporating the concept of TTI, Graph C creates a unitless horizontal axis. The resulting curve is normalized along both dimensions and can serve as the fundamental diagram of reliability, referred to throughout this report as a *cumulative TTI curve.* The cumulative TTI curve is a cumulative distribution function of the TTI for a given time-slice (TTI-CDF). This curve has a series of properties that are useful indicators of reliability. Perhaps the most fundamental is that the closer the curve's shape is to a vertical line at TTI = 1.0 (the minimum x-value), the more reliable is the facility it describes. This reliability indicates that there is little difference in the travel times between trips of the shortest duration and trips of the longest duration, and that the travel time index for even longer trips is close to 1.0. Graph D illustrates this principle.

Evaluating Reliability: Indicators

Although the travel time distribution serves as a defining diagram for reliability, simpler quantitative measures are usually the backbone of analysis. Figure 1.2 shows a sample TTI-CDF

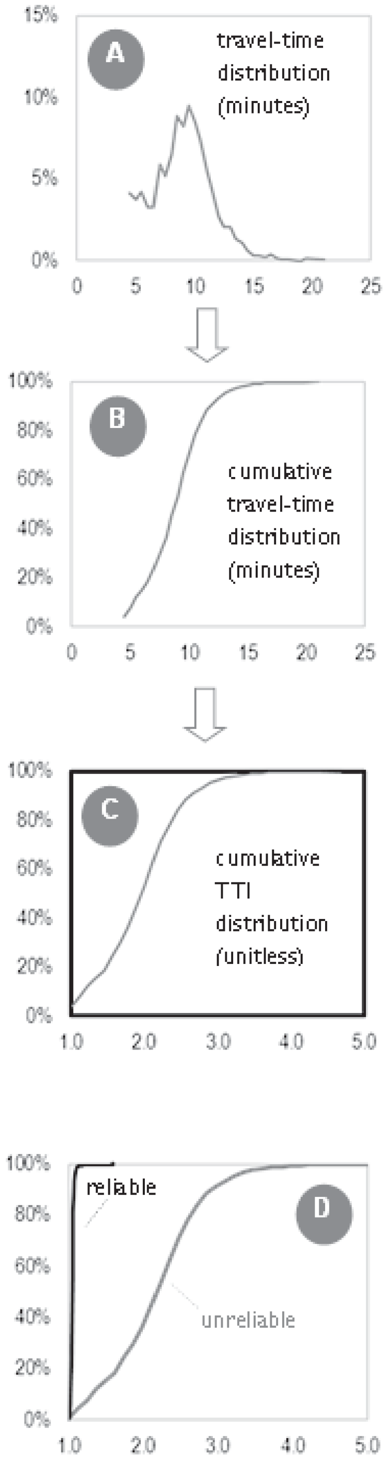


Figure 1.1. Cumulative TTI curve.

of 1-year hourly time-slices from an actual highway segment; analysts desire numerical measures to distinguish among these curves. To be useful, such measures must describe an aspect of the travel time distribution (most often, its shape). The following discussion begins with the two fundamental descriptors of any statistical distribution (mean and variance)

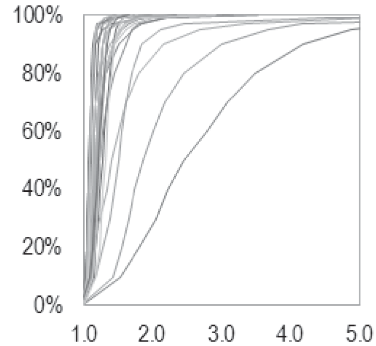


Figure 1.2. Sample 24-h cumulative TTI curves.

and extends the discussion to other measures that have been derived from the travel time distribution.

Mean-Based Measures

Certain measures, such as mean TTI and the lateness index, relate to the mean of the travel time distribution.

Mean Travel Time Index

The mean of the TTI distribution (TTI_{mean}) can hint at a facility's reliability. A facility with a TTI_{mean} of 1.1 would probably be considered "reliable," and a facility with a TTI_{mean} of 2.0 would probably be considered "unreliable." Strictly speaking, the term *undesirable* is more appropriate than unreliable when referring to a "bad" TTI_{mean}, because the mean generally conveys no information regarding the shape (variability) of the distribution. However, research has shown that reliability decreases with increasing congestion, to the extent that at least one report (*1*) has concluded that "reliability is a feature or attribute of congestion." One could imagine a distribution such as the one in Figure 1.3, in which travel times are "reliably" clustered around an undesirable TTI (in this case, 2.0). However, such distributions are not common in reality, because when a facility nears its capacity, delay values are very volatile, and so the

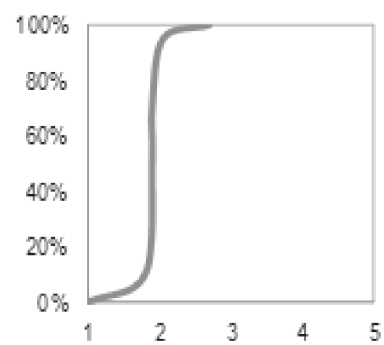


Figure 1.3. Reliable but undesirable distribution: a theoretical construct.

cumulative curve generally leans forward like the outer curves in Figure 1.2. Thus, more "unreliable" curves will have higher TTI_{mean} values.

Lateness Index

A slight enhancement to TTI_{mean} acknowledges the distinction between travel time and delay. The difference between a user's actual travel time and desired free-flow travel time across a segment (or for an entire trip) can be said to be equivalent to that user's delay. Since a TTI of 1.0 equates to free-flow conditions, delay can be thought of as proportional to TTI − 1. This quantity, although trivial to calculate once a TTI has been calculated, has a physical analog, as illustrated in Figure 1.4. Because the cumulative TTI curve is unitless, the shaded area in Figure 1.4 is equal to $TTI_{mean} - 1$. A suggested name for this quantity is the lateness index (LI). If the LI is multiplied by the total number of vehicles in the time-slice (V) and the free-flow travel time of the segment (TT_{FF}), then the result is the total delay experienced by all vehicles, as shown by Equation 1.1:

$$\text{Total delay} = \text{LI} \times V \times \text{TT}_{FF} \tag{1.1}$$

Variance-Based Measures

Certain measures relate to the variance of the travel time distribution, as described below.

Variance and Standard Deviation

A distribution's variance and standard deviation are indicators of how far the distribution spreads out. As such, these measures are more powerful descriptors of reliability than the mean, because reliability is primarily concerned with variability.

Variance about the mean (σ) is calculated as shown by Equation 1.2 (assuming a continuous distribution), with TTI_i representing the ith percentile TTI and n representing 100% (the maximum y-value on the cumulative TTI curve). The standard deviation is given by $\sqrt{\sigma}$.

$$\sigma = \frac{1}{n}\int_{i=0}^{n} (\text{TTI}_i - \text{TTI}_{\text{mean}})^2 \, di \tag{1.2}$$

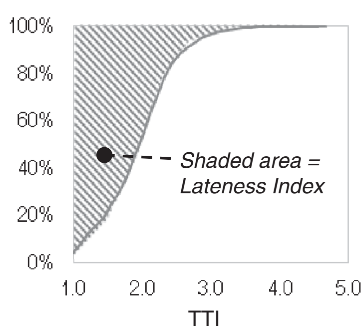


Figure 1.4. Lateness index.

Semivariance

Although calculating the variance about the mean (as shown by Equation 1.2) is fairly common statistical practice, describing how travel times differ from the mean is potentially not as useful as describing how they differ from the ideal. Therefore, the concept of the semivariance (σ_r) has been used. Statistically, semivariance can be described as the second moment of the travel time distribution about the minimum, as shown by Equation 1.3:

$$\sigma_r = \frac{1}{n}\int_{i=0}^{n} (\text{TTI}_i - r)^2 \, di = \int_{i=0}^{100\%} (\text{TTI}_i - 1)^2 \, di \tag{1.3}$$

In this case, r, the reference value from which deviation is calculated, is set to 1.0, the minimum possible (or ideal) TTI. The value of n is set to 100%, echoing the upper limits of the cumulative TTI distribution.

Figure 1.5 illustrates the difference in the values used to build the variance $(\text{TTI} - \text{TTI}_{mean})^2$ and semivariance $(\text{TTI} - 1)^2$. The variance curve is constructed from the cumulative TTI curve by calculating $(\text{TTI} - c)^2$ for each percentile p, the difference between TTI_p and the vertical line $y = \text{TTI}_{mean}$, and then squaring that difference. The semivariance curve is constructed the same way, except using the vertical line $y = 1$. Thus, σ and σ_r can be computed by taking the area to the left of the appropriate

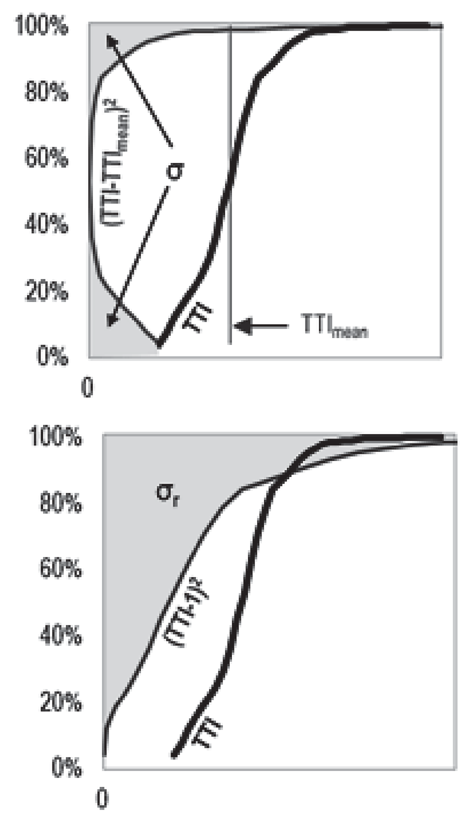


Figure 1.5. (Top) variance and (bottom) semivariance buildups.

curves (shaded in the figure). With curves that lean forward, σ_r will always be much larger than σ. A "reliable but undesirable" TTI distribution like the one shown in Figure 1.3 would have a very low σ (indicating low variability with respect to the mean), but a higher σ_r (indicating high variability from the ideal).

LI (Figure 1.4) and the semivariance provide roughly the same information about the cumulative TTI curve: the former is a summation of TTI − 1, and the latter is a summation of $(TTI - 1)^2$. The semivariance places disproportionate emphasis on larger deviations, and therefore may better gauge reliability.

Single-Point and Regime Indices

Several measures used in reliability analysis relate to points or regions on the cumulative TTI curve. Figure 1.6 illustrates these measures in the context of the cumulative curve. Generally, such measures have been developed for values well above the median TTI, because the upper portion of the cumulative curve yields the most information about reliability.

Planning Time Index

The planning time index (PTI) is equal to the 95th percentile TTI. Its name derives from the idea that it represents the total time travelers should allow to ensure on-time arrival 95% of the time.

Misery Index

The misery index (MI) represents the average of the highest 5% of travel times ("the worst day of the month"). On the cumulative TTI curve, it is equal to the average *x*-coordinate in the circled area in Figure 1.6. MI may be especially useful in characterizing rural reliability, for which even a relatively small number of very delayed trips can be a source of major frustration for motorists. One approximation for the MI is $TTI_{97.5\%}$; this approximation assumes roughly linear behavior of the cumulative TTI curve above the 95th percentile.

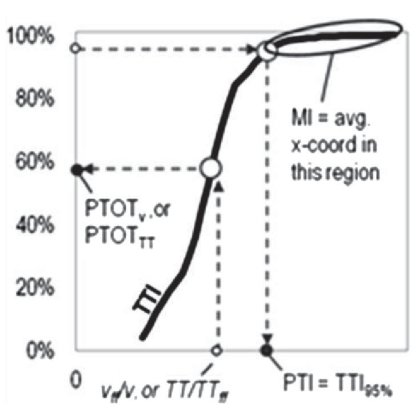


Figure 1.6. Single-point and regime indices.

Percentage of Trips on Time

Percentage of trips on time (labeled "PTOT" in Figure 1.6) essentially works in the reverse direction of the PTI and MI, in effect specifying a target TTI and then extracting the corresponding percentile from the cumulative TTI curve. Percentage of trips on time represents the percentage of trips completed within a certain speed or time range, such as the percentage of trips that arrived on time with a speed of 45 mph or greater or the percentage of trips that arrived on time with a TTI of 1.5 or less.

Overall, no single point (or small region) in the travel time distribution is a comprehensive descriptor of reliability. For example, Figure 1.7 illustrates two curves with identical PTI values ($TTI_{95\%}$) but very different behavior in the upper tails. Nevertheless, values in the upper percentiles can certainly convey a sense of how much the cumulative distribution leans forward.

Curvature Indices

Several reliability measures are built on ratios that describe aspects of the curvature of the cumulative curve. Figure 1.8a illustrates these measures in relation to the cumulative curve.

Buffer Index

The buffer index (BI) describes how much the cumulative TTI curve "leans forward" beyond the mean or median. The term *buffer* indicates the extra time that travelers should add to their average travel times to ensure on-time arrival (buffer time equals planning time minus average time). Like the PTI, BI hinges on the 95th percentile TTI, but it uses a ratio involving either the mean (Equation 1.4) or the median (Equation 1.5):

$$BI_{mean} = (TTI_{95\%} - TTI_{mean})/(TTI_{mean}) \tag{1.4}$$

$$BI_{50\%} = (TTI_{95\%} - TTI_{50\%})/(TTI_{50\%}) \tag{1.5}$$

Recent research has raised doubts about the use of the BI as a primary reliability metric for tracking trends in reliability

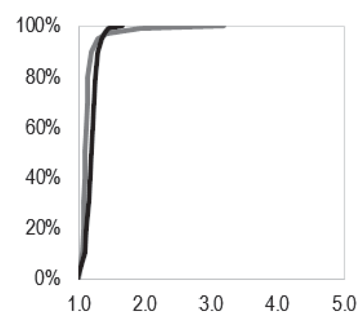


Figure 1.7. Identical PTIs.

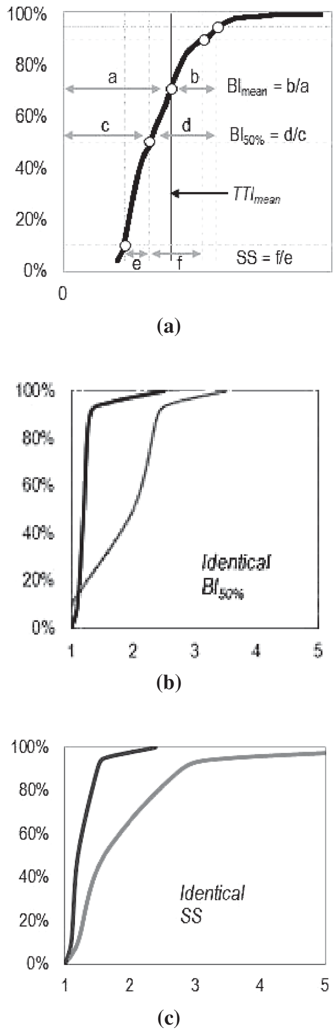


Figure 1.8. Curvature indices.

due to its erratic and unstable nature (*1*). Treatments that tend to uniformly decrease travel times (rather than affecting only the extremes) can result in counterintuitive BIs (falsely indicating reliability degradations when conditions are actually improving). However, BI remains useful as a secondary metric.

Skew Statistic

The skew statistic (SS) is a measure of symmetry in the travel time distribution, calculated as a ratio of 40th percentile TTI ranges on either side of the median TTI, as shown by Equation 1.6:

$$SS = (TTI_{90\%} - TTI_{50\%})/(TTI_{50\%} - TTI_{10\%}) \tag{1.6}$$

Measures such as the BI and skew statistic, although providing information about the shape of the travel time distribution, do not provide sufficient information about the desirability of the distribution. Figure 1.8, b and c, illustrates that very different distributions can have identical BIs and skew statistics, respectively.

Summary of Reliability Indicators

As discussed above, it is not merely unreliability, but undesirable unreliability that must be quantified. One can analogize to capacity-based analyses, in which an index (level of service) gets worse as an undesirable quantity (delay) gets larger. Similarly, it is logical for a reliability-based index to increase as undesirable variability increases. Measures of area around the cumulative TTI distribution such as LI and semivariance best exhibit this behavior. Curvature indices (BI, skew statistics) do not do so reliably. Point measures cannot always tell the full story. The cumulative curve itself is the best metric of reliability. By studying its shape in a given situation, the analyst can determine which supplemental measures are appropriate.

No universal standard has yet been developed for acceptable values of any reliability index. When standards are developed, they will likely vary for different physical environments (e.g., large metropolitan area, smaller metropolitan area, rural area) and differing facility types (e.g., freeway or arterial).

Comparing Reliability

The cumulative travel time distribution and its properties can be used to compare reliability conditions on a facility before and after the implementation of a proposed improvement. For example, the cumulative TTI graph in Figure 1.9 shows data from an actual freeway segment before and after a reliability improvement (a ramp-metering implementation). The shaded area is equal to the differences in the LI and can be termed the "lateness reduction," which, when multiplied by the segment's volume (V) and free-flow travel time (TT_{FF}), translates to an overall delay reduction. Thus, the area between the TTI curves before and after improvement is proportional to the overall delay reduction.

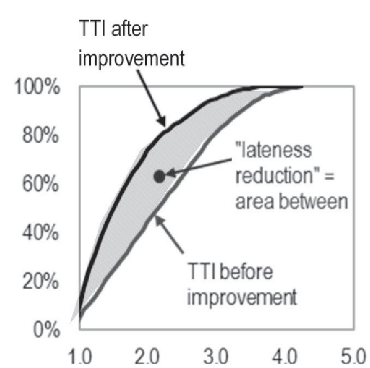


Figure 1.9. Delay reduction.

The delay reduction illustrated in Figure 1.9 can further be translated into economic terms by using the monetary value of time. Research has shown that motorists directly value reductions in travel time variability, leading to the idea that a similar graph could be constructed for some measure of variance and translated into economic terms by using a monetary value of reliability.

Predicting Reliability

Essential to reliability's application as a measure of highway system performance is the ability to forecast the effect of an improvement strategy (or even a "do-nothing" strategy) on a facility's near- and long-term reliability. Recent research has broken new ground in correlating reliability measures to predictable attributes or events, proposing a series of equations for predicting reliability based on three highway and environment attributes (*1*):

- A general measure of highway congestion (ratio of demand to capacity)
- A measure of temporal–spatial impacts of incidents and work zones (lane hours lost)
- A measure of precipitation amount (rain and snow)

As explained in Chapter 4, these predictive formulas are the foundation for most of the operational analysis work of Project L07.

Organization of the Report

The remainder of this report is organized as follows.

Chapter 2 describes the original research objective and scope and how they grew over the life of the project to address additional research needs. It explains the evolution of the research approach based on the reliability models developed by another SHRP 2 project that preceded this effort. Chapter 2 also briefly summarizes the three research products in addition to this final report: the Design Guide, Analysis Tool, and Dissemination Plan.

Chapter 3 describes the various sources of data used to develop the methods, models, and default values found in the products of this research. Data sources include the reliability models developed by SHRP 2 Project L03; the traffic operational databases available from Seattle, Washington, and Minneapolis–St. Paul, Minnesota; crash data from the same cities; weather data for stations around the United States from the National Climactic Data Center; a literature review; and interviews with state highway agency staff. Chapter 3 also lists all the nonrecurrent congestion design treatments considered in this research.

Chapter 4 explains in mathematical terms how the predicted TTI distributions for a section of freeway during a specific time of day can be used to calculate operational benefits of design treatments in terms of reduced total delay and improved reliability. The mechanics of mapping the effects a given treatment has on operations into the reliability model variables (demand-to-capacity ratio, lane hours lost, rainfall, and snowfall) are presented in detail.

Chapter 5 presents the methodology for estimating the direct and indirect safety benefits of design treatments for nonrecurrent congestion, so that they can be accounted for in the benefit–cost analysis. The direct benefits include the reduction in crash frequency or severity expected as a result of changes to lane width, shoulder width, or other geometric features related to base capacity as indicated by *Highway Capacity Manual* procedures; and other roadway and roadside design features that may affect driver behavior, likelihood of a crash, or severity of a crash. The research team found a relationship between crash frequency and level of service, described in Chapter 4. This relationship predicts the indirect safety benefits expected as a result of an improvement in level of service.

Chapter 6 describes the methodology for placing the operational and safety benefits estimated in Chapter 4 and Chapter 5 in economic terms to compute the net present benefit of a design treatment. In addition, a procedure is described for determining a treatment's net present cost and computing the benefit–cost ratio.

The research team developed a "reasonableness test" to evaluate the outputs provided by the procedures described in this report and implemented in the Analysis Tool described in Chapter 2. The test was used to initiate an iterative quality control process of implementing changes based on test results and then retesting. This effort is described in Chapter 7.

Major findings from all phases of the research are summarized in Chapter 8. These conclusions came not only from the literature and meetings with highway agencies, but also from the development of the various models and procedures presented in this report. They include insights gained by the research team through careful study of previously developed reliability measures and visual presentation of those measures. Chapter 8 concludes with recommendations for how the results of this research might be implemented by highway agencies.

CHAPTER 2

Research Approach

Chapter 1 presented and discussed the six primary sources of nonrecurrent congestion. Research in SHRP 2 Project L07 addressed these sources of unreliable travel times by identifying various design treatments that may be considered by highway agencies to reduce nonrecurrent congestion. Initially, the scope of Project L07 focused on five of the six sources of nonrecurrent congestion:

- Traffic incidents
- Work zones
- Traffic control devices
- Special events
- Demand fluctuations

However, during the first year of the project, SHRP 2 expanded the scope of Project L07 to address weather as a cause of nonrecurrent congestion and to include design treatments that may be used to reduce nonrecurrent congestion related to snow and ice and other weather-related events.

Research Objective and Scope

The objectives of this research were to (1) identify the full range of possible roadway design features used by transportation agencies on freeways and major arterials to improve travel time reliability and reduce delays from key causes of nonrecurrent congestion, (2) assess their costs and operational and safety effectiveness, and (3) provide recommendations for their use and eventual incorporation into appropriate design guides.

The research focused on geometric design treatments to reduce nonrecurrent congestion. However, some of these treatments are broader in scope than just geometric design. For example, some include traffic control, incident management, or motorist services. That is, some treatments of interest are directly related to geometric design, but other treatments have an important, but indirect, relationship to geometric design (e.g., they are supported by geometric design features).

Three separate analyses of the design treatments were conducted to achieve the research objectives. The primary analysis was a traffic operational assessment that estimated a distribution of travel times on a freeway segment with a specific set of geometric and operational characteristics and then estimated the expected change in the distribution of travel times after the implementation of a treatment. This shift in the distribution of travel times provides information about delay savings and improved reliability of the roadway as a result of implementing a design treatment. A secondary analysis of the safety implications of using the design treatments was also conducted. Although this analysis considered direct safety benefits of treatment installation, it focused on the indirect benefits associated with reduced nonrecurrent congestion. The research team explored the relationship between crash frequency by severity and level of service to develop a model for predicting the reduction in crashes due to a reduction in nonrecurrent congestion. These two analyses were then used as inputs, along with a user-defined treatment cost, into the benefit–cost analysis of treatments.

The traffic operational analysis methodology developed in this research was intended to build from work completed in SHRP 2 Project L03. However, the products of Project L03 did not precisely meet the needs of the Project L07 analysis. The next section describes the evolution of the research approach for this analysis. Chapter 4 of this report provides a detailed description of the traffic operational analysis, Chapter 5 describes the safety analysis, and Chapter 6 describes the benefit–cost analysis.

Evolution of Research Approach for Traffic Operational Analysis

The research team's original concept for Project L07 was that delay measures (i.e., vehicle hours of delay) for specific design treatments for nonrecurrent congestion would be obtained through a combination of the following:

- Direct calculation of performance measures from field data
- Deterministic analysis techniques (primarily those of the *Highway Capacity Manual* [*2*])
- Microscopic traffic simulation
- Qualitative methods, when necessary

During the development of the work plan for Project L07, the research team for another SHRP 2 Reliability project (Project L03, Analytic Procedures for Determining the Impacts of Reliability Mitigation Strategies) anticipated that their reliability models would estimate vehicle hours of delay and then translate those delay estimates into reliability measures. Specifically, it was anticipated that the models being developed by Project L03 could be very useful in translating the Project L07 delay measures into reliability measures, as follows:

Treatment → ΔEvent or physical or traffic characteristics → ΔDelay → ΔReliability

Therefore, this approach was recommended in the work plan for Project L07.

However, the models that were actually developed by Project L03 and presented in the final report of that project estimate reliability measures directly without first quantifying vehicle hours of delay. Thus, Project L03 took a somewhat different approach to modeling than its team originally anticipated. Furthermore, as the Project L07 research team studied the Project L03 relationships and began to apply them to specific design treatments, some constraints and boundary conditions of the Project L03 models became apparent. In particular, the Project L03 models are most applicable to urban freeways in major metropolitan areas, but the scope of Project L07 included rural and small- and medium-sized urban areas, as well. In addition, the Project L03 models are most applicable to peak periods, but Project L07 focused on nonrecurrent congestion, which occurs at any time of the day or night. Therefore, the research team revised the approach for Project L07 as follows:

- Reliability measures for design treatments were determined using the Project L03 models directly for the conditions to which these models apply; this generally included time-slices (i.e., portions of the day) in which the demand-to-capacity (*d/c*) ratio was greater than or equal to 0.8.
- Delay measures for the effect of design treatments were developed for a broader range of traffic conditions than those to which the Project L03 models apply (i.e., including traffic conditions representative of off-peak conditions in major urban areas, peak and off-peak conditions in small- to medium-sized urban areas, and peak and off-peak conditions for rural areas). These conditions generally include time-slices in which the *d/c* ratio is less than 0.8. The delay measures were developed with simulation modeling for each design treatment to which simulation modeling was applicable.

Thus, the operational effects of the design treatments were initially quantified with a combination of reliability measures from the Project L03 models and delay measures from simulation modeling.

Ideally, however, the L07 research team and SHRP 2 hoped that reliability models could be developed for the full range of *d/c* ratios; that is, for congested and uncongested periods. Furthermore, the Project L03 models included a variable to account for rainfall ($R_{0.05''}$), but the models did not account for snow conditions. To address these and other issues, SHRP 2 approved an extension of Project L07 to further develop and refine the analytical framework and the spreadsheet-based Analysis Tool that were developed earlier in the project. Specifically, the extension of the project focused on the following:

- Further development of the models to address the effects of snow and ice on the traffic operational effectiveness of design treatments
- Further development of the models to address the effects of multihour incidents on the traffic operational effectiveness of design treatments
- Analysis of existing data to improve the applicability of reliability models for time periods with $d/c < 0.8$
- Verification of the reasonableness of evaluation results for design treatments obtained with the spreadsheet-based Analysis Tool

Products of the Research

In addition to the documentation of the research found in this final report, the research plan included the development of two key products: a design guide for nonrecurrent congestion treatments, and an information dissemination plan. Through the course of conducting the traffic operational analysis and applying reliability models to assess the traffic operational effectiveness of design treatments, the research team also developed a spreadsheet-based treatment analysis tool. This analysis tool, which is accompanied by a user guide, represents a third key product in the research.

Design Guide

The Design Guide catalogs the design treatments considered in this research, providing planners, designers, operations engineers, and decision makers with a toolbox of possible options for addressing nonrecurrent congestion through design treatments. The Guide begins with an introduction to nonrecurrent congestion and reliability, a discussion of the six main causes of nonrecurrent congestion, and a basic explanation of how the reduction of delay and the improvement of reliability can be valued in economic terms. Next, the design treatments that were considered in this research are presented with a decision tree that assists the user in narrowing the full list of design treatments to a shorter list that may be appropriate for further consideration and evaluation. Following the decision tree, the design treatments are cataloged, and relevant information is provided in the following categories:

- Treatment description and objectives
- Typical applications
- Design criteria
- How treatment reduces nonrecurrent congestion
- Factors affecting treatment effectiveness
- Factors affecting treatment cost
- References

The Design Guide's final chapter includes examples of existing implementations of many of the design treatments. These examples are brief and include information available from internet searches, interviews with agency staff, and the research team's own experience through field visits to various treatment installations. The intent of this chapter is to provide users with information about the cost, successes, and challenges experienced by agencies who have implemented a treatment in the past and to provide a starting point from which the user can seek additional information from the agency that implemented the treatment.

The Design Guide, in its entirety, is meant to serve as a primary reference for planners, designers, operations engineers, and decision makers interested in reducing nonrecurrent congestion and improving reliability on their freeways. The document does not have to be read completely; its main function is to serve as a catalog of nonrecurrent congestion treatments that the user can browse to find information about specific treatments of interest. It is anticipated that the Guide will help users identify a few treatments that may be applicable to a specific roadway of interest and to investigate further by using the Analysis Tool discussed in the next section.

Analysis Tool

The Analysis Tool was developed to allow highway agencies to analyze and compare the effects of a range of design strategies on a given highway segment using the analytical procedures developed in this research. Analysts can input data about the highway (e.g., geometrics, volumes, crash totals), and the tool computes delay and reliability indicators resulting from various design treatments, further translating those results into life-cycle costs and benefits. The tool, shown in Figure 2.1, is a VBA interface overlaying a Microsoft-based Excel 2007 spreadsheet.

The tool is designed to analyze a generally homogenous segment of a freeway (typically between successive interchanges). Based on user-input data, the tool calculates base reliability conditions. The user can then analyze the effectiveness of a variety of treatments by providing fairly simple input data regarding the treatment effects and cost parameters. As outputs, the tool predicts cumulative travel time index (TTI) curves for each hour of the day, from which other reliability variables are computed and displayed. The tool also calculates cost-effectiveness by assigning monetary values to delay and reliability improvements and comparing these benefits to the expected costs over the life of each treatment. The tool is interactive, in that results are immediately updated and displayed as inputs are changed.

The tool is designed to be used in conjunction with two companion documents: this report and the Project L07 Design Guide. It is also supported by an annotated user guide. The tool is the first of its kind, and reliability analysis is still in its infancy. Although this tool (and its successors) will become more sophisticated in the future, it is nevertheless a comprehensive approach to applying the principles developed in Project L07.

The tool interface is divided into three parts, as shown in Figure 2.1:

- *Site inputs.* The user enters data regarding location (e.g., segment name, length), geometry (e.g., number of lanes, lane widths, grade), demand (hourly demand, peak hour factors, and truck percentages for a typical 24-h day), special event information (hourly volume percentage increase and event frequency for up to nine events), work-zone information (work-zone feature and days active for up to nine work zones), precipitation data, and incidents (annual crash and incident totals by severity and type).
- *Treatment data and calculations.* The user enters specific data regarding each selected treatment's effects, including percentage of incidents reduced by type and the effects of the treatment on average incident duration. The user also enters treatment construction and annual maintenance costs. The tool calculates and displays the treatment's benefits (operational and safety), and displays net present benefit and benefit–cost ratio as measures of cost-effectiveness.
- *Results.* For each hour of the day, the tool graphs the five reliability variables that are inputs to the TTI prediction models (see Chapter 4), the treated and untreated cumulative TTI curves for each hour, and a series of reliability measures of effectiveness.

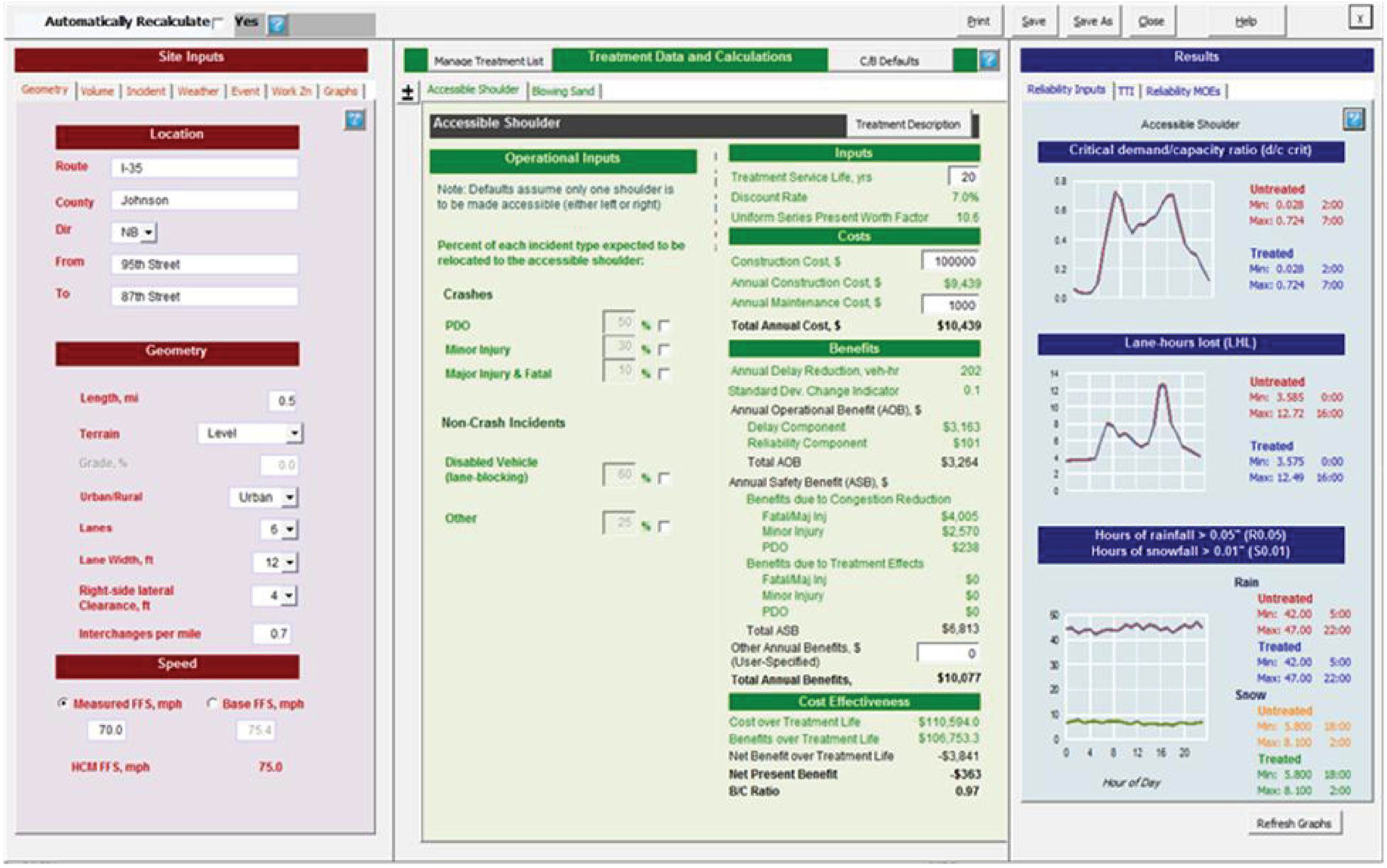


Figure 2.1. Project L07 spreadsheet Analysis Tool: user interface.

Dissemination Plan

From the initial development of the Project L07 scope of work, it was determined that a successful dissemination plan needed to be developed. Such a plan would provide a strategic approach to disseminating the results of the research. The objectives of the Dissemination Plan were the following:

- Increase awareness of Project L07's research findings, including the benefits and value of the Design Guide and Analysis Tool within the transportation community.
- Spur the adoption and integration of the Design Guide and Analysis Tool into policies and standard practice within the transportation community.

A dissemination plan has been developed and submitted to SHRP 2. The plan includes a four-pronged approach to disseminating the research results:

1. Provide clear and distinct messages outlining what the products are and how they add value to the target audience.
2. Engage partnerships to help reach a broader audience and add credibility to the research recommendations and products.
3. Deliver effective training for prospective implementers on how to use the products.
4. Offer a strategy for what target audiences should do with the information.

The strategic dissemination of Project L07's research results requires outreach to multiple stakeholder groups, with careful consideration of each group's values and needs. The Dissemination Plan addresses the following:

- Types of organizations that need to receive information about the research results
- Types of individuals within those organizations who are the target audience for information dissemination
- Types of media and materials that should be used to reach those individuals
- Methods for managing and monitoring the success of the information dissemination effort

The Dissemination Plan also accounts for the overarching activities of the SHRP 2 marketing program. The effectiveness of these activities is, however, dependent on their concurrent implementation with the overall marketing efforts of SHRP 2.

CHAPTER 3

Data Collection and Documentation of Current Design Practice

The research team conducted a number of activities aimed at (1) gathering and synthesizing information on existing and promising design treatments and (2) collecting data that could be used to evaluate the effectiveness of these treatments at reducing delay due to nonrecurrent congestion. These activities included the following:

- Obtaining of travel time reliability models from SHRP 2 Project L03
- Assembly of traffic operational, crash, and weather databases from various sources
- Review of completed and ongoing research related to design treatments to address travel time reliability, delay, and nonrecurrent congestion
- Initial contact, through e-mail and telephone, with highway agencies to obtain relevant information about design treatments in use, or considered for use, to reduce nonrecurrent congestion
- Focus groups with select highway agencies to gather details and insights about design treatments in use
- Workshops with highway agencies to gather details and insights about design treatments to address nonrecurrent congestion due to weather events
- Meetings with highway agencies to obtain detailed information about design treatments
- Development of a list of design treatments for evaluation

This section of the report summarizes each of these activities.

Reliability Models from SHRP 2 Project L03

SHRP 2 Project L03, Analytic Procedures for Determining the Impacts of Reliability Mitigation Strategies, developed models to predict several points along the annual travel time distribution of a highway segment for a given time-slice (*1*). (A time-slice can be a 1-h or multihour period, and Project L03's models apply to nonholiday weekdays only.) The travel time distribution is essentially the fundamental descriptor of reliability, from which other reliability indicators of interest (e.g., buffer time, planning time) can be readily derived. For the purposes of both Project L03 and Project L07, travel time is most conveniently represented by the travel time index (TTI), which is defined as the ratio of actual travel time on a segment to the free-flow travel time. The Project L03 models quantify the effect of incidents and work zones on reliability by predicting several percentiles of the TTI distribution on the basis of three key variables:

- Lane hours lost due to incidents and work zones, which is calculated as the average number of lanes blocked per incident (or work zone) multiplied by the average duration per incident (or work zone) multiplied by the total number of incidents (or work zones) during the time-slice and study period of interest.
- Critical demand-to-capacity ratio, which is defined as the ratio of demand to capacity during the most critical hour of the time-slice and study period.
- Hours of rainfall exceeding 0.05 in. during the time-slice and study period.

Project L03 developed these relationships for various time-slices of the day over an extended study period. These time-slices included peak hour, peak period, midday (the 2-h period from 11:00 a.m. to 1:00 p.m.), and weekday (all 24 h). Because nonrecurrent congestion can occur at any time of the day, Project L07 needed a relationship that covered each of the 24 h of the day. The only Project L03 model that could quantify reliability for an hourly time-slice was the peak hour model, so this was the model used in Project L07 (at least to evaluate the effectiveness of design treatments during congested conditions). Reliability models more applicable to uncongested conditions were developed in Project L07, as discussed in Chapter 4 of this report.

Table 3.1. Coefficients Used in Project L03 Reliability Models: Peak Hour (2)

n (percentile)[a]	j_n	k_n	l_n
10	0.07643	0.00405	0.00000
50	0.29097	0.01380	0.00000
80	0.52013	0.01544	0.00000
95	0.63071	0.01219	0.04744
99	1.13062	0.01242	0.00000

[a] The coefficients used to calculate the mean TTI are 0.27886 for j_n, 0.01089 for k_n, and 0.02935 for l_n.

The Project L03 relationships have the following general functional form, as shown in Equation 3.1:

$$\mathrm{TTI}_{n\%} = e^{(j_n \mathrm{LHL} + k_n \mathrm{dc}_{\mathrm{crit}} + l_n R_{0.05''})} \tag{3.1}$$

where

$\mathrm{TTI}_{n\%}$ = nth percentile TTI value;
LHL = lane hours lost;
$\mathrm{dc}_{\mathrm{crit}}$ = critical demand-to-capacity ratio;
$R_{0.05''}$ = hours of rainfall exceeding 0.05 in.; and
j_n, k_n, l_n = coefficients for nth percentile (see Table 3.1).

Table 3.1 shows the coefficients used to calculate each TTI percentile, as derived by Project L03 for peak hour data. The resulting TTI percentile values can be plotted as cumulative TTI curves, as illustrated in Figure 3.1, which shows 24 cumulative curves for each hour of the day from actual field data for a freeway in Minnesota. As an example of interpreting such curves, the darkened curve (representing the worst, i.e., most unreliable, hour of the day) has a 50th percentile TTI of 2.3, signifying that 50% of the vehicles that travel through this roadway segment during that hour spend more than 2.3 times the amount of time that it would take to traverse this segment under free-flow conditions.

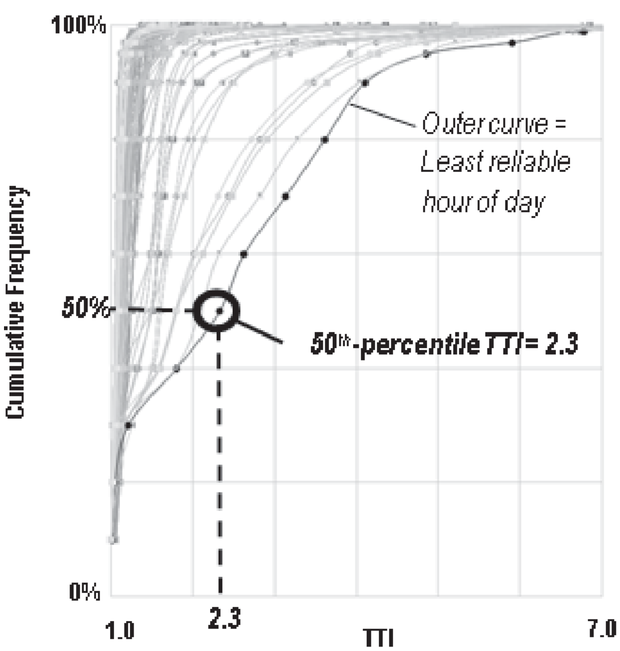


Figure 3.1. Cumulative TTI distribution per hour of day.

Assembly of Databases

Traffic operational, crash, and weather data were obtained for use in several analyses in the research, including the following:

- Analysis of traffic operational and crash data to determine a relationship between safety and congestion for use in evaluating design treatments
- Analysis of traffic operational and weather data to develop reliability models that accounted for both rain and snow conditions
- Analysis of traffic operational data to develop reliability models that were applicable to less congested conditions (i.e., $d/c < 0.8$)

Each of these databases is described below.

Traffic Operational Data

Three years (2005 to 2007) of traffic operational data were obtained from the SHRP 2 Project L03 research team from freeways in two metropolitan areas: Seattle, Washington, and Minneapolis–St. Paul, Minnesota. The sites in Seattle included two to four directional lanes of travel and represented 200 mi of directional freeway segments. The sites in Minneapolis–St. Paul included two to five directional lanes of travel and represented 410 mi of directional freeway segments. Each station for which traffic volume and speed data were available included detectors in each lane across one direction of travel on a freeway. The original detector data collected at each station on the freeways consisted of 5-min volume data per travel lane and 5-min average speed data per travel lane.

A decision was reached to exclude from the study all data in the Minneapolis–St. Paul metropolitan area after the I-35W bridge collapse on August 1, 2007. Although this period might have been interesting (because volumes changed dramatically on many freeway segments), the changed driving conditions were new to many drivers and the Minnesota Department of Transportation (DOT) made many modifications to specific roadways to increase base capacity; thus, this time period would likely include unusual flow conditions.

Crash Data

Crash data for each directional freeway segment were obtained through the Highway Safety Information System. The crash

Source: © Microsoft Streets and Trips.

Figure 3.2. U.S. weather stations with data available.

data included all mainline freeway crashes that occurred within the limits of each roadway section of interest during the study period. Crash severity levels considered in the research were the following:

- Total crashes (i.e., all crash severity levels combined)
- Fatal-and-injury crashes
- Property-damage-only crashes

Weather Data

The research team obtained 10 years (2001 through 2010) of hourly precipitation data across the United States from the National Climactic Data Center (NCDC). From the NCDC website, the research team downloaded quality-controlled local climatological data databases for all stations within the United States.

The databases used in the analysis were the "Precip," "Hourly," and "Station" files. The Precip database contained a record of total precipitation at each station for each hour of the day. The hourly database included a variable (WeatherType) that indicated specific weather conditions and that was used to classify precipitation as either rain or snow. The Station file listed information about the stations that reported during a given month, including station number, station name, latitude, and longitude. A subset of 387 weather stations (those with a World Meteorological Organization designation) was selected for use in the research. These 387 stations are depicted in Figure 3.2.

Review of Completed and Ongoing Research

The types of design treatments applicable to nonrecurrent congestion, and the objectives of those design treatments, were identified through a review of completed and ongoing research, technical articles, vendor literature, conference proceedings, highway agency and technical association websites, internet search engine query results, and direct contacts with highway agencies. The research team looked for the following information:

- Design treatments being used or considered for use in reducing congestion
- The applicability of design treatments to nonrecurrent congestion
- Design guidelines or standards for treatments
- Implementation policies and practices for treatments
- Cost estimates of treatments
- Traffic operational effectiveness of treatments
- Safety effectiveness of treatments
- Other key information on the use of design features for nonrecurrent congestion

The research team also documented international experience with design treatments to reduce nonrecurrent congestion.

The overall results of the literature review can be summarized as follows:

- There is substantial information about the effects of design treatments on recurrent congestion.
- There is substantial information about the effects of intelligent transportation system strategies on nonrecurrent congestion.
- There is only limited information about the effects of design treatments on nonrecurrent congestion.

This lack of information about the effects of design treatments on nonrecurrent congestion showed a clear need for research on this topic.

Initial Contacts with Highway Agencies

The research team contacted highway agencies to obtain relevant information about design treatments in use, or considered for use, to reduce nonrecurrent congestion. The research team contacted 20 state highway agencies to obtain this information. Telephone interviews were then conducted with knowledgeable engineers in the most promising agencies. The highway agencies that were contacted included those in the following states:

- Arizona
- California
- Florida
- Georgia
- Illinois
- Indiana
- Maryland

- Massachusetts
- Michigan
- Minnesota
- Missouri
- New Jersey
- New York
- North Carolina
- Ohio
- Tennessee
- Texas
- Virginia
- Washington
- Wisconsin

Telephone interviews were also conducted with the following agencies:

- Florida's Turnpike Enterprise
- New York State Thruway Authority
- Port Authority of New York and New Jersey

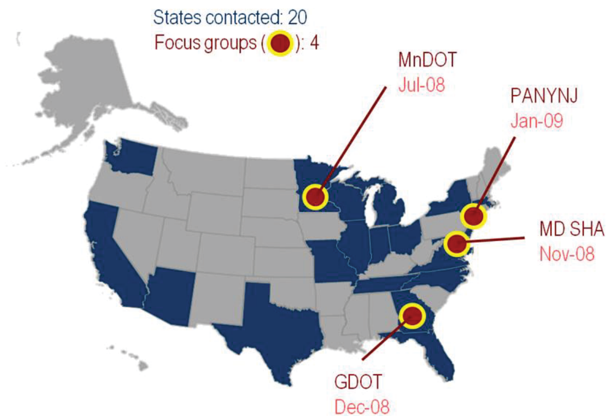


Figure 3.3. Darkly-shaded states were contacted by e-mail or telephone; Minnesota DOT (MnDOT), the Port Authority of New York and New Jersey (PANYNJ), Maryland State Highway Authority (MD SHA), and Georgia DOT (GDOT) participated in focus groups.

During the telephone interviews, the research team discussed with each highway agency (1) design treatments being used or considered for use in reducing congestion, (2) the applicability of those treatments to nonrecurrent congestion, (3) whether the agency would be willing to participate in a focus group as part of the research, and (4) whether the agency had any suitable projects or sites for evaluation in the research.

Focus Groups with Highway Agencies

The research team gathered details and insights about design treatments identified through initial contacts with highway agencies by conducting focus groups in the following four metropolitan areas with active congestion reduction programs:

- Minneapolis–St. Paul: Minnesota DOT
- Atlanta: Georgia DOT
- Baltimore–Washington, D.C.: Maryland DOT
- New York City–Newark: Port Authority of New York and New Jersey

The four metropolitan areas were selected for conducting focus groups because they represented diverse geographic regions of the country and because, based on the telephone interviews conducted by the research team, they were actively involved in using design treatments to address delay caused by nonrecurrent (as well as recurrent) congestion.

Figure 3.3 shows the states that were contacted by e-mail or telephone, as well as the metropolitan areas where focus groups were conducted.

Each focus group meeting consisted of a 2-day visit. On the first day of the visit, the research team met with several highway agency staff experienced in geometric design, traffic operations, traffic management, or maintenance to review implemented or planned projects to reduce nonrecurrent congestion. Issues discussed and the types of questions asked included the following:

- What design treatments have been used in your region to reduce nonrecurrent congestion?
- Is nonrecurrent congestion considered and addressed in the design phase of new projects? If so, how?
- Who is involved in the decision making for implementing a nonrecurrent congestion mitigation strategy? What agencies and departments are involved? Whose responsibility is it? Who takes the lead?
- What treatments have been used to address recurrent congestion that could be considered for use in nonrecurring congestion situations?
- What treatments are considered promising but have not yet been tried in your region?
- How does your agency decide whether to implement a treatment in additional locations?

For each treatment identified by the highway agency as having been implemented, the research team asked the following questions:

- What information is available about the traffic operational effectiveness, safety effectiveness, and cost of the treatment?

- Are there any design policies and guidelines for the treatment?
- What application criteria are used to determine when and where the treatment should be installed?
- What difficulties and challenges have been encountered in implementing the treatment?
- What are the perceived advantages and disadvantages of the treatment?
- What historical data are available concerning deployment of the treatment to address nonrecurrent congestion?
- Are any crash data available to compare sites with and without the treatment?

On the second day of the visit, the research team made field visits to several implemented treatments in the area.

Workshops with Highway Agencies to Discuss Weather Treatments

Because design treatments that address weather events were added to the scope of work several months after Project L07 began, they were not initially considered in the same depth as other design treatments. To provide further consideration of weather-related treatments, the research team held two workshops with highway agencies to ensure that the list of design treatments related to weather events was complete and that the current state of knowledge concerning the effectiveness of such treatments was fully documented. The primary focus of these workshops was on design treatments related to winter weather events; however, other weather-related events were also considered.

The first workshop was held in Kansas City, Missouri, and included weather experts from the following highway agencies and organizations:

- Florida DOT
- Missouri DOT
- City of Kansas City, Missouri
- City of Overland Park, Kansas
- City of West Des Moines, Iowa
- McHenry County, Illinois

The local agencies participated in person; the others participated by phone. The workshop included a discussion of the types of weather events experienced by each participating agency that led to nonrecurrent congestion. The primary focus of the workshop was on design treatments that participating agencies have used to reduce the impact of weather events on congestion.

The second workshop was held in Minnesota with key Minnesota DOT staff with expertise in winter weather events and design treatments to address those events, as well as knowledge in geometric design, traffic operations, and maintenance. Participants included Minnesota DOT staff from the Minneapolis–St. Paul metropolitan area, as well as from various rural districts.

Design treatments related to weather events that were identified during the two workshops included the following:

- Snow fences
- Road weather-information systems
- Anti-icing systems
- Flood warning systems
- Fog detection systems
- Wind warning systems
- Contraflow for hurricane evacuation
- Road closure

Meetings with Highway Agencies to Obtain Detailed Information About Design Treatments

As part of the focus groups (discussed above), the research team documented highway agency experience with existing treatments and gathered basic information about their design and application. However, to conduct traffic operational assessments of design treatments, more detailed treatment information was needed. Several additional visits to highway agencies were made to gather more detailed information about treatments that had been implemented. In particular, the research team met with three highway agencies to obtain detailed information about many geometric design treatments, but with a particular emphasis on crash investigation sites, for which more information was clearly needed. These agencies are the following:

- *Minnesota DOT.* The research team met with geometric designers, traffic engineers, and maintenance personnel; the meeting included representatives from Minnesota DOT's incident response program (FIRST) and the Minnesota Highway Patrol.
- *Illinois DOT.* The research team met with traffic engineers responsible for traffic operations and incident management in the Chicago metropolitan area.
- *Wisconsin DOT.* The research team met with traffic engineers responsible for traffic operations and incident management in the Milwaukee metropolitan area.

Crash investigation sites varied greatly among the three agencies. For example, most of the sites that Minnesota DOT has constructed in recent years would more appropriately be considered emergency pulloffs, because they have been implemented where shoulders are no longer available due to the shoulders being converted to travel lanes. The emergency pulloffs serve several purposes, including crash investigation and enforcement, but they were primarily constructed to accommodate inoperable vehicles and other emergencies that would otherwise be accommodated by a shoulder. Illinois DOT's crash investigation sites range in size and design and have been installed in a variety of locations, including along the right side of the freeway (beyond the shoulder), inside the median, on ramps, and underneath overpasses.

The research team gathered as much detailed information as possible about the use of crash investigation sites so that the traffic operational effectiveness of such sites could be estimated most accurately. Key information obtained during the highway agency visits included the following:

- Typical number of lanes blocked during an incident
- Policy about moving crashes to an emergency pulloff or crash investigation site
- Types of crashes moved
- Percentage of crashes moved
- Average time between when a crash occurs and when it gets moved
- Average reduction in lane hours lost when a crash is moved
- Typical dimensions and design of an emergency pulloff or crash investigation site
- Typical signing at an emergency pulloff or crash investigation site
- Cost issues with constructing an emergency pulloff or crash investigation site

The research team used this information to develop input variables for the models developed in Project L03, and any related simulation models that were developed, to estimate the impact of crash investigation sites on nonrecurrent congestion.

During the highway agency visits, detailed information was obtained for other treatments, as well, including the following:

- Alternating shoulders
- Bus pulloffs
- Bus shoulders
- Designated bus lanes
- Dynamic shoulders
- Emergency pulloffs
- Emergency traffic operations plan
- Extra-height median barriers
- Flood control systems
- Geometric improvements to alternate routes
- Glare screens
- Median crossovers
- Movable barriers
- Ramp closure
- Ramp metering
- Reversible lanes
- Traffic signal improvements
- Ramp metering
- Variable speed limits

List of Design Treatments

Using the results of the initial contacts and follow-up focus groups with highway agencies, the research team identified a list of design treatments to be further assessed in the research. The factors that were used as the basis for deciding which design treatments should be considered for assessment included the following:

- Treatment is used (or can be used) for nonrecurrent congestion.
- Treatment supports one or more of the objectives. (An objective is how a treatment is used to reduce nonrecurrent congestion; e.g., it reduces the duration of the incident.)
- Operational effectiveness of the treatment is promising.
- Safety effectiveness of the treatment is promising.
- Cost of the treatment is low to moderate.
- Treatment has broad application potential.
- Treatment is a strong candidate for inclusion in the Project L07 Design Guide.

A particular design treatment did not have to meet all of these criteria to be selected for further assessment in the research. Only the first two criteria—that the treatment addressed nonrecurrent congestion and that it supported one or more of the objectives—were mandatory. All of the design treatments selected met not only those two mandatory criteria, but several of the other criteria, as well.

Table 3.2 presents the specific design treatments that were selected for assessment in the research. The design-related treatments are those that are highway design features or function through changes in highway design features. Specifically, the nonrecurrent congestion design treatments are implemented through physical changes in the highway design that have a direct influence on traffic flow. For example, providing a paved shoulder or widening an existing shoulder is a nonrecurrent congestion design treatment. Some treatments, like shoulders, affect traffic flow at all times; others, like portable incident screens, affect traffic flow only at times when the treatment is deployed or in operation. The secondary treatments are not intrinsically highway design features, but they have secondary implications related to highway design. For example, ramp metering is a traffic control strategy rather than a highway design feature; however, the implementation of ramp metering has implications for the design of ramps

Table 3.2. Candidate Design Treatments Considered in the Research

Category	Nonrecurrent Congestion Design Treatment	Secondary Treatment
Medians	Median crossovers	—
	Movable traffic barriers	
	Gated median barrier	
	Extra-height median barriers	
	Mountable or traversable medians	
Shoulders	Accessible shoulder	—
	Drivable shoulder	
	Alternating shoulder	
	Portable incident screens	
	Vehicle turnouts	
	Bus turnouts	
Crash investigation sites	Crash investigation sites	—
Right-of-way edge	Emergency access between interchanges	—
Arterials and ramps	Ramp widening	—
	Ramp closure	
	Ramp terminal traffic control	
	Ramp turn restrictions	
Detours	Improvements to detour routes	—
Truck incident design considerations	Runaway truck ramp	—
Construction	Reduce construction duration	
	Improved work site access and circulation	
Animal–vehicle collision design considerations	Wildlife fencing, overpasses, and underpasses	—
Weather	Snow fences	Fog detection
	Blowing sand treatment	Roadway weather-information system
	Anti-icing systems	Flood warning system
		Wind warning system
Lane types and uses	—	Contraflow lanes for emergency evacuation
		Contraflow lanes for work zones
		HOV lanes and HOT lanes
		Dual facilities
		Reversible lanes
		Work-zone express lanes

(continued on next page)

Table 3.2. Candidate Design Treatments Considered in the Research (continued)

Category	Nonrecurrent Congestion Design Treatment	Secondary Treatment
Traffic signals and traffic control	—	Traffic signal preemption
		Queue jump and bypass lanes
		Traffic signal improvements
		Signal timing systems
		Ramp metering and flow signals
		Temporary traffic signals
		Variable speed limits and speed limit reduction
Technology	—	Electronic toll collection
		Overheight vehicle detection and warning systems
Emergency response notification	—	Reference location signs
		Roadside call boxes

Note: HOV = high-occupancy vehicle; HOT = high-occupancy toll; — = not applicable.

that highway agencies must understand in order to use this treatment effectively.

In developing the list of design treatments in Table 3.2, careful consideration was given to whether to include design treatments whose sole or primary function is to increase the base capacity of the roadway. Such treatments primarily address recurrent congestion, which was outside the scope of the Project L07 research. Design treatments for interchanges such as ramp braiding, adding collector–distributor roads, and adding auxiliary lanes were not included in Table 3.2 because they reduce congestion solely or primarily by increasing base capacity. Although any design treatment that increases the base capacity of the roadway will reduce both recurrent and nonrecurrent congestion, the primary purpose of such treatments is to reduce recurrent congestion. Furthermore, once implemented to reduce recurrent congestion, such treatments are available at any time to reduce nonrecurrent congestion, with no intervention required by the highway agency or traffic management center when incidents, demand fluctuations, or special events occur.

Two exceptions were made to the general principle that design treatments that increase base capacity should be excluded from the research scope. First, if a design treatment functions to relieve nonrecurrent congestion not only by increasing base capacity, but also in other ways, it was considered within the research scope. For example, adding shoulders or widening existing shoulders on a roadway increases the base capacity of the roadway, but also provides a storage area for vehicle breakdowns, a safe stopping place for service assistance patrols, increased flexibility for work-zone operations, as well as the potential for use when needed as an additional through lane. Second, some design treatments provided primarily to reduce recurrent congestion have the potential for use to address nonrecurrent congestion, but require an explicit decision by the highway agency or traffic management center. Such treatments were also considered within the scope of the research. Examples of such design treatments include reversible lanes and HOV lanes, which could be used to accommodate special event traffic or to route traffic around a major incident.

Technology-related treatments, such as changeable message signs and demand detection systems (loop detectors or video detection), have a key role in reducing nonrecurrent congestion, but were not considered design treatments for purposes of this research. However, technology-related treatments were considered within the scope of the research to the extent that they support a design treatment. For example, changeable message signs may be used to communicate detours, ramp closures, or ramp turning restrictions to drivers, or in conjunction with using reversible lanes for special events. Demand detection systems may be used to detect an incident so that an appropriate treatment may be implemented more quickly.

The design treatments in Table 3.2 served as the basis for the traffic operational, safety, and life-cycle benefit–cost analyses presented in the following three chapters.

CHAPTER 4

Traffic Operational Assessment

Overview

The cumulative distribution function of the travel time index (TTI-CDF) curve was introduced in Chapter 1 as the fundamental diagram from which reliability statistics can be computed. Chapter 1 presents methods to predict values along the TTI-CDF of a freeway segment based on fundamental traffic flow and physical and environmental characteristics. Chapter 1 further demonstrates how predicted TTI-CDFs for treated and untreated conditions can be used to calculate the operational benefits of a given design treatment.

Figure 4.1 illustrates the process developed in Project L07 to calculate these benefits and indicates which sections in Chapter 4 cover different aspects of the process.

Prediction of Cumulative TTI Curve

Background

Research for SHRP 2 Project L03 (*1*) developed predictive relationships for several percentiles on the cumulative TTI curve for a given time-slice as a function of key parameters:

- A general measure of highway congestion (ratio of demand to capacity)
- A measure of temporal–spatial impacts of incidents and work zones (lane hours lost)
- A measure of precipitation amount over a specified period (rain)

The Project L03 models were developed for several time-slices (peak hour, peak period, midday, and weekday). The Project L07 research team was most interested in single-hour time-slices, which allow development of predictions for each of the 24 h of the day. This approach allows the consideration of all incidents or events that may potentially result in nonrecurrent congestion (not just those that occur during already congested periods) and the aggregation of hourly operational measures into meaningful daily and annual statistics that can be used in economic analysis. Only the peak hour models in Project L03 are based on a single-hour time-slice. The Project L07 research team revised and extended these models to apply to nonpeak (uncongested) hours, as well.

The Project L07 research team also extended the Project L03 models to include a snow variable, in addition to the rain variable already considered.

Model Variables

The models that were used and enhanced to predict cumulative TTI percentiles are based on four primary variables:

- d/c—Demand-to-capacity ratio
- LHL—Lane hours lost due to incidents and work zones
- $R_{0.05''}$—Hours of rainfall exceeding 0.05 in.
- $S_{0.01''}$—Hours of snowfall exceeding 0.01 in.

Each variable is described in detail below.

Demand-to-Capacity Ratio

The d/c variable, which indicates the level of day-to-day congestion on the highway facility, is defined as the ratio of demand (d) to capacity (c) for a given highway segment over a given time-slice.

Calculating Demand

Demand is defined as weekday nonholiday demand during the 30th-highest hour of the year during a given time-slice. Demand differs from volume in that demand represents all motorists who would travel on a section given unconstrained capacity during a given period (everyone who wanted to travel on the freeway section), and volume is equal to the observed or counted vehicles during the same period (everyone who

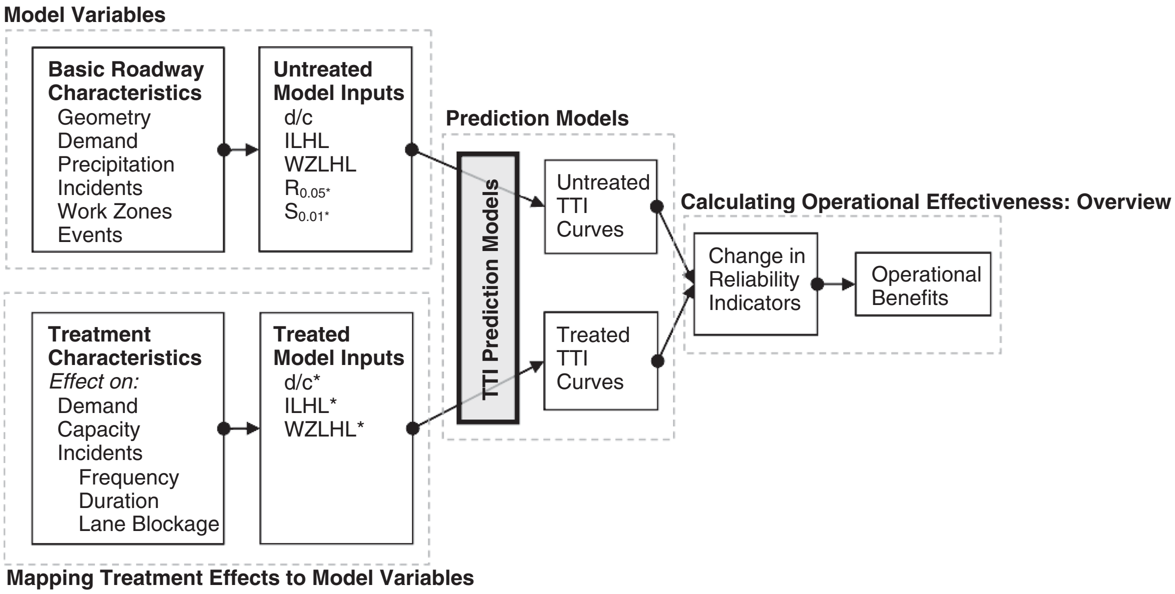


Figure 4.1. Calculation of operational effectiveness.

actually traveled on the freeway section). Therefore, when demand is less than capacity, volume equals demand. Three methods of computing demand are described below:

1. If observed volume data for each nonholiday weekday hour for the entire year (roughly 250 counts for each of the 24 h) are available, then the analyst can directly select the 30th-highest volume (v_{30}) for each of the 24 h. For all uncongested periods, demand equals volume (v_{30}). For periods when demand may exceed capacity, volumes can be converted to demand by using one of the following two methods:
   a. If volume and speed data are available in 5-min increments, the analyst can use the method developed in SHRP 2 Project L03 to compute demand (*1*). The procedure identifies consecutive 5-min periods during which the mean speed drops below a congested level (typically the 35- to 45-mph range). Demand is estimated by extrapolating the flow rate just before the onset of congestion, resulting in an assumed queue, and then further assuming that the queue begins to dissipate midway through the congested period. Adjustments may be needed at the end of the congested period to ensure a smooth cumulative demand curve.
   b. If volume and speed data are not available in 5-min increments, and the analyst has only the hourly volumes to work with, it is recommended that the analyst make field observations of the times when congestion begins and ends on the facility and then estimate or measure the evolution of the queue during that congested period. The total number of vehicles queued upstream of the segment during the hour (q) can be assumed to be equal to the residual demand; thus, an approximation for the demand is given by Equation 4.1:

$$d = v_{30} + q \tag{4.1}$$

2. If, as is often the case, the analyst has a single-day or multiday count, the following procedure can be used to compute the volume for the 30th-highest hour. Most state departments of transportation (DOTs) tabulate factors that allow conversion of average daily traffic (ADT) to annual ADT (AADT) as a function of the month of the year and day of the week on which the volumes were collected (seasonal and daily factors). The typical calculation is shown by Equation 4.2:

$$\mathrm{AADT} = \mathrm{ADT} \times f_{\mathrm{month},m} \times f_{\mathrm{day},d} \tag{4.2}$$

where $f_{\mathrm{month},m}$ and $f_{\mathrm{day},d}$ are the factors to convert month m to the average month and day d to the average weekday, respectively. These factors can be used to convert the observed volume (v_{obs}) to approximate the 30th-highest-hour volume (v_{30}) for a given hour by using Equation 4.3:

$$v_{30} = v_{\mathrm{obs}} \left(\frac{f_{\mathrm{month,MAX}} f_{\mathrm{day,AVG}}}{f_{\mathrm{month},m} f_{\mathrm{day},d}} \right) \tag{4.3}$$

where $f_{\mathrm{month,MAX}}$ represents the factor for the maximum month of the year, and $f_{\mathrm{day,AVG}}$ represents the average of the factors for all five weekdays. Thus, Equation 4.3 essentially sets v_{30} equal to the average day in the peak month (for the given hour). Allowing for some peak holiday and weekend travel, this is a good approximation for the 30th-highest hour.

In some cases, the analyst may be aware that, due to extreme volume fluctuations or the presence of major traffic generators, the 30th-highest hour is higher than Equation 4.3 would suggest. One way to illustrate this issue is to consider special events. If the volume is known to be heavier than the calculated v_{30} on more than 30 days (due to special events), v_{30} can be set equal to the volume of the 30th highest of these event days. The above method for including events is recommended as an initial procedure. To convert v_{30} to demand for use in the *d/c* equation, Procedure 1.b above using field observations ($d = v_{30} + q$) is recommended for any periods that experience congestion.

Because the frequency of events and demand surges varies from facility to facility and from city to city, incorporating event-related demand surges into reliability calculations is a complex endeavor that has not been fully addressed in previous research.

3. If the analysis is based on future volumes, a travel-demand forecasting model can be used to predict demand. However, as the forecasted demand may be the mean and not the 30th-highest hour, the monthly and daily factors described above may also need to be applied.

All demand volumes should be converted to passenger car equivalents by using heavy-vehicle percentages and passenger car–equivalent factors from the *Highway Capacity Manual* (HCM), Chapter 11 (*2*).

Calculating Capacity

To calculate capacity, procedures from Chapter 11 of the HCM 2010 are used to derive the free-flow speed for the freeway section by using geometric information about the section. The free-flow speed is converted into a lane capacity and multiplied by the number of lanes to give total segment capacity in vehicles per hour. Capacity may vary throughout the day. For example, a reversible lane may be available only at certain times of day, or a shoulder may be used as a lane only during peak periods. In dividing the day into 24 separate 1-h periods, the analyst must ensure that the capacity values for each hour account for these effects if present.

Effects of Long-Term Work Zones

This research distinguishes between short- and long-term work zones. Short-term work zones (lasting 7 days or less) are considered nonrecurrent congestion, and as such are evaluated as part of the work-zone lane hours lost (WZLHL) variable discussed later in this section. Long-term work zones (longer than 30 days) do not comfortably fit into the nonrecurrent congestion category, and therefore a different analysis approach must be used. A long-term work zone essentially establishes a "new normal" base capacity that should be used to test against any potential improvements affecting nonrecurrent congestion in the work zone (such as emergency pulloffs). Medium-term work zones, lasting between 8 and 29 days, currently fall into an analytical gray area. They typically provide WZLHL values that fall outside the TTI prediction models discussed in Chapter 3; the analyst is cautioned to carefully weigh analysis results for work zones of these durations.

Calculating *d/c*

The adjusted hourly volumes (d^*) are divided by the capacity (c) for each hour to calculate an individual *d/c* value for each of the 24 h of the day. (The asterisk [*] indicates the variable as affected by the treatment.)

Lane Hours Lost Due to Incidents and Work Zones

This variable is a quantitative measure of the extent, duration, and frequency of incidents and work zones—items that temporarily reduce freeway capacity. LHL is defined as the sum of incident lane hours lost (ILHL) and work-zone lane hours lost (WZLHL) for a time-slice. Conceptually, LHL represents the effective number of lanes blocked due to all incidents and work zones during the time-slice, multiplied by the average blockage time for each incident and work zone. It correlates to the nonrecurrent capacity decreases attributable to these causes.

The two components of LHL, ILHL and WZLHL, are defined and described below.

Lane Hours Lost Due to Incidents

ILHL is defined as the effective number of lanes blocked due to all incidents occurring during a time-slice, multiplied by the average blockage time for each incident type. ILHL is calculated as shown by Equation 4.4:

$$\text{ILHL} = \frac{1}{60}\sum_i \left(N_{\text{incidents},i}\, N_{\text{blocked},i}\, T_{\text{incidents},i}\right) \tag{4.4}$$

where

$N_{\text{incidents},i}$ = number of incidents of type i during the time-slice;

$N_{\text{blocked},i}$ = average number of lanes blocked per incident of type i; and

$T_{\text{incidents},i}$ = average duration of incident of type i (min).

Each element of the ILHL equation is discussed below.

Incident Type. Project L07 considers six incident types. The first three are crashes categorized by the standard crash severity scale, and the last three are noncrash incidents.

- Crash—property-damage-only (PDO)
- Crash—minor injury
- Crash—major injury or fatal

Table 4.1. Suggested (Default) Proportions for Noncrash Incidents

Percentage of incidents that are crashes	22%
→ Inferred ratio of noncrash incidents to crash incidents	3.545
Proportion of noncrash incidents by type	
Disabled—non-lane-blocking	71%
Disabled—lane-blocking	18%
Other noncrash incidents	11%

- Disabled vehicle—non-lane-blocking (shoulder)
- Disabled vehicle—lane-blocking
- Other noncrash incidents

Many items can potentially be included in the “other noncrash incidents” category, such as roadway obstructions and message-board gawking. The Project L07 research team included gawking (rubbernecking) as an opposite-direction incident in this category. In other words, a slowdown caused by gawking at an incident in the opposite direction is itself considered an incident. The literature is inconclusive on whether gawking is included in the typical definition of an incident, but the L07 research team has found this categorization necessary to ensure that the analysis methodology is applicable for evaluating treatments that mitigate this type of gawking.

Calculating $N_{\text{incidents}}$. Calculating the number of incidents of each type *i* during the time-slice is generally straightforward for crashes, but typically less so for noncrash incidents. Often, an agency will have detailed information on crashes, but very little data on noncrash incidents. If such data are unavailable, the values in Table 4.1 are suggested as defaults. The first two values in the table are based on Project L03 research (*1*), and the relative proportions of noncrash incidents are based on Project L07 discussions with highway agencies.

Diurnal Distribution of $N_{\text{incidents}}$. As $N_{\text{incidents},i}$ must be calculated for each hour of the day, the analyst must distribute annual incidents over 24 h. If crash data are not available by hour of day, or data are being forecasted, the following procedures can be used:

- *Diurnal distribution of crashes.* Project L07 developed a relationship between crash rates and traffic density, the level of service measure for freeways (see Chapter 5). This relationship can be used to distribute crashes between hours of the day over the 24-h period.

  By using methods discussed in HCM, Chapter 11 (see HCM Exhibit 11-3) (*2*), the average operating speed (S) for each hour is calculated on the basis of the hourly vehicular volume (or demand) (V) and free-flow speed. The density D for each hour i can then be determined by using Equation 4.5:

  $$D_i = \frac{V_i}{S_i} \tag{4.5}$$

  Using this density, the analyst can then use the L07 crash–density relationship presented in Table 4.2 to predict a crash rate (per million vehicle miles traveled) for that hour of the day for each crash type.

  Although these relationships could be used to predict an hourly number of crashes, it is assumed that the analyst already knows the observed site-specific crash totals, so the individual hourly predictions are used only to prorate the known annual crash total. Thus, for each of the three crash types, $N_{\text{incidents}}$ is calculated for each hour of the day (i) as shown by Equation 4.6:

  $$N_{\text{incidents},i} = \frac{C^*_{H,i}}{\sum_{j=1}^{24} C^*_{H,j}}(C_D) \tag{4.6}$$

Table 4.2. Predicted Crash Rate as a Function of Traffic Density (Project L07)

	Crash Rate As Function of Density (D_i)					
		If $20 \le D_i \le 78$				
		$C = a_1D_i^3 + a_2D_i^2 + a_3D_i + a_4$				
Crash Severity	**If $D_i < 20$**	a_1	a_2	a_3	a_4	**If $D_i > 78$**
Fatal or major injury ($C_{\text{fatal or}}$ C_{major})	0.25	-1.795×10^{-5}	0.00264	–0.0842	1.022	2.02
Minor injury (C_{minor})	0.25	-1.795×10^{-5}	0.00264	–0.0842	1.022	2.02
Property-damage only (C_{PDO})	0.55	-3.01×10^{-5}	0.00444	–0.1301	1.614	4.20
Total crashes (C_{tot})	0.80	-4.80×10^{-5}	0.00708	–0.2143	2.636	6.22

Note: The development of this relationship is discussed in Chapter 5.

where

$C^{*}_{H,i}$ = predicted total crash frequency for hourly time-slice i from Project L07 crash–density relationship for given crash severity (see Table 4.2); and

C_D = observed total crash frequency for all hours of the day over the entire year for given crash severity (based on crash history data).

Other crash prediction methods are becoming available that also incorporate the influence of roadway geometric features. For example, NCHRP Project 17-45 includes crash prediction guidance for geometric design elements such as shoulder width, lateral clearance, and presence and type of outside barriers. As these methods become more widely adopted, analysts can use them, coupled with procedures from the AASHTO *Highway Safety Manual* (5), to enhance the methodology presented above.

- *Diurnal distribution of noncrash incidents.* A reasonable assumption is to distribute noncrash incidents throughout the day in proportion to the hourly volumes, as shown by Equation 4.7:

$$N_{\text{incidents},i} = \frac{V_{H,i}}{\sum_{j=1}^{24} V_{H,j}} (I_D) \qquad (4.7)$$

where $V_{H,i}$ is the traffic volume for hour i, and I_D is the daily incident total for a given incident type (see default percentages in Table 4.1).

Calculating N_{blocked}. To calculate $N_{\text{blocked},i}$ (the average number of lanes blocked per incident for each incident type i), the recommended procedure is to use the ratio of the blocked and unblocked capacities to calculate an effective equivalent number of blocked lanes, as shown by Equation 4.8:

$$N_{\text{blocked},i} = N_L(1 - R_{\text{cap},i}) \qquad (4.8)$$

where

$N_{\text{blocked},i}$ = average number of lanes blocked per incident of type i;

N_L = number of lanes on the facility (one direction); and

$R_{\text{cap},i}$ = capacity for incident of type i.

To calculate $R_{\text{cap},i}$, the recommended procedure is to adapt ratios from HCM, Exhibit 22-6 (2), which provides freeway capacity reduction proportions for various types of incidents. The HCM exhibit is based on a combination of incident types and lane blockages; therefore, Project L07 developed a procedure to convert the percentage of freeway capacity available (from the HCM exhibit) to the capacity reduction ratio for the six incident types used in this research.

Table 4.3 includes recommended values for $R_{\text{cap},i}$ and the assumptions used to develop them from the HCM.

Calculating $T_{\text{incidents}}$. To determine $T_{\text{incidents},i}$ (the average duration for an incident of type i), the analyst can use local data.

Table 4.3. $R_{cap,i}$ Values Used to Calculate $N_{blocked,i}$ for ILHL

No. of Freeway Lanes (One Direction)	Crash Type			Noncrash Incident Type (Disabled Vehicle)		
	PDO	Minor Injury	Major Injury and Fatal	Non-Lane-Blocking	Lane-Blocking	Other
2	0.67	0.58	0.16	0.95	0.34	0.83
3	0.73	0.64	0.29	0.99	0.48	0.87
4	0.77	0.69	0.38	0.99	0.57	0.89
5	0.80	0.74	0.48	0.99	0.64	0.90
6	0.84	0.78	0.56	0.99	0.70	0.92
7	0.86	0.81	0.62	0.99	0.74	0.93
8	0.89	0.84	0.66	0.99	0.77	0.94
Values above are adapted from HCM, Exhibit 10-17, based on assumed conversions below from blockage type to incident type						
Shoulder disablement	0%	0%	0%	100%	0%	50%
Shoulder crash	72%	59%	5%	0%	0%	39%
1 Lane blocked	26%	28%	35%	0%	96%	10%
2 Lanes blocked	2%	10%	45%	0%	3%	1%
3 Lanes blocked	0%	3%	15%	0%	1%	0%

Table 4.4. Incident Duration Default Values, $T_{incident}$

Incident Type		Incident Duration (min)
Noncrash	Lane-blocking	20
	Non-lane-blocking	26
	Other noncrash incidents	28
Crash	PDO	28
	Minor injury	40
	Major injury and fatal	45

If local data are unavailable, the default values in Table 4.4 are suggested by the Project L07 research team, based on interviews and focus groups with highway agencies. However, incident duration is heavily dependent on emergency response and clearance times and certain highway agency policies, so these values should be adjusted based on local agency practices and actual experience wherever possible.

Treatments or actions that shorten the incident timeline should, by definition, reduce incident duration. Typical incident timelines are illustrated in Figure 4.2 for two cases: (1) when an incident is left in place (blocking traffic lanes) until cleared, and (2) when an incident is moved to the shoulder for further responder work before clearing the incident—converting to a rubbernecking incident until it is completely cleared. In both cases, what is referred to throughout this document as "incident duration" is measured from the point marked "incident occurrence" to the point marked "incident cleared." As the figure suggests, the timing of several events—including responder notification (e.g., via 911), initial response time (time to arrive on scene), and others not specified (such as response protocols once on scene)—can heavily influence incident duration. The incident duration further influences overall delay, because longer incidents result in longer queues and therefore queue discharge times (all else being equal). The time required for queue discharge is not included in the incident duration as defined for this project.

Lane Hours Lost Due to Work Zones

WZLHL is a measure of the effective number of lanes blocked due to all short-term work zones occurring during a time-slice, multiplied by the effective amount of time they will be active during the time-slice. WZLHL is calculated as shown by Equation 4.9:

$$\text{WZLHL} = \left(1 - \frac{c_{\text{WZ}} N_{\text{lanes,WZ}}}{c N_{\text{lanes}}}\right) N_{\text{days}} \tag{4.9}$$

where

c_{WZ} = per lane capacity of the work zone (passenger cars per hour per lane [pcphpl]; HCM 2010, Chapter 10 suggests a default capacity of 1,600 pcphpl, with adjustments due to lane width and ramp presence);

$N_{\text{lanes,WZ}}$ = number of open lanes through the work zone;

c = per lane capacity of the freeway section before establishment of the work zone (this should be the same value used in the d/c_{crit} calculation);

N_{lanes} = number of lanes on the segment before establishment of the work zone; and

N_{days} = number of days the work zone is active during the time-slice.

For the purposes of Project L07, long-term work zones were not considered as nonrecurrent congestion. If a work zone will be in place for a relatively long period of time (e.g., more than 30 days), rather than being considered as part of the WZLHL calculation, it should be factored into base capacity assumptions for the highway segment of interest (see the previous *d/c* discussion).

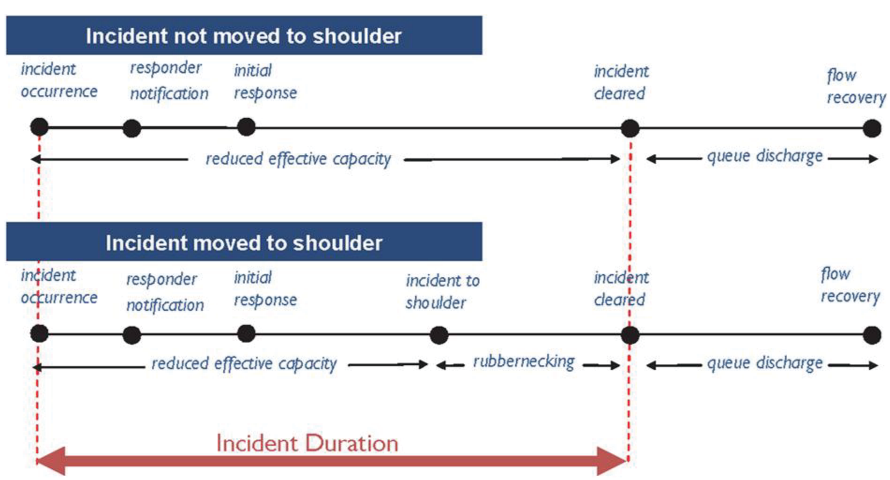


Figure 4.2. Typical incident timelines.

If more than one short-term work zone is expected to occur on a highway segment during the time-slice, individual WZLHL values are computed for each work zone and then summed.

Hours of Rainfall Exceeding 0.05 in.

$R_{0.05''}$ is the measure, for a particular time-slice, of the total number of hours in which 0.05 in. or more of rainfall is observed.

Because data on hourly rainfall over long periods of time are not readily available to transportation analysts, the research team has assembled default data that can be applied by users of the Project L07 methods. The research team developed these data on the basis of 10 years (2001 through 2010) of hourly precipitation data at 387 weather stations across the United States (see Figure 4.3). The spreadsheet tool described in Chapter 2 under Products of the Research incorporates the rainfall database to automatically determine a value for $R_{0.05''}$ when any city in the United States is selected.

Hours of Snowfall Exceeding 0.01 in.

$S_{0.01''}$ is the measure, for a particular time-slice, of the total number of hours in which snowfall exceeding trace amounts (0.01 in.) is observed.

The original Project L03 models, which did not contain snowfall data, were enhanced by the Project L07 research team to account for snow. The snowfall data were obtained from the same weather stations as were the rainfall data for $R_{0.05''}$. Appendix A describes the development of the snow model extension.

Prediction Models

The four variables described in the previous section (d/c, LHL, $R_{0.05''}$, and $S_{0.01''}$) are the independent variables used in the travel time reliability models to predict various TTI percentiles, or points along the cumulative TTI curve. These models are designed to be applied for single-hour time-slices. The development of these models is described in Appendix A.

Source: © Microsoft Streets and Trips.

Figure 4.3. U.S. weather stations used to determine $R_{0.05''}$ and $S_{0.01''}$.

The reliability models used in Project L07 to estimate the effectiveness of design treatments at reducing nonrecurrent congestion and, thus, improving travel time reliability are expressed as shown in Equation 4.10:

$$\mathrm{TTI}_n = \begin{cases} \mathrm{TTI}_{\mathrm{NP},n} \times e^{(c_n R_{05''} + d_n S_{01''})} & \text{for } d/c \le 0.8 \\ \dfrac{\mathrm{TTI}_{\mathrm{NP},n}}{N_{\mathrm{days}}} \times \left[N_{\mathrm{NP}} + V_{\mathrm{FF}} \left(\dfrac{R_{05''}}{c1_n V_{\mathrm{FF}} + c2_n \mathrm{TTI}_{\mathrm{NP},n}} + \dfrac{S_{01''}}{d1_n V_{\mathrm{FF}} + d2_n \mathrm{TTI}_{\mathrm{NP},n}} \right) \right] & \text{for } d/c > 0.8 \end{cases} \qquad (4.10)$$

where

TTI_n = predicted nth percentile TTI;
$\mathrm{TTI}_{\mathrm{NP},n}$ = nonprecipitation portion of $\mathrm{TTI}_n = e^{(a_n d/c + b_n \mathrm{LHL})}$;
LHL = LHL due to incidents and work zones;
d/c = demand-to-capacity ratio;
$R_{05''}$ = number of hours in time-slice with rain exceeding 0.05 in.;
$S_{01''}$ = number of hours in time-slice with snow exceeding 0.01 in.;
N_{days} = number of hours in time-slice (365);
N_{NP} = number of hours in time-slice with no precipitation (i.e., $N_{\mathrm{days}} - R_{05''} - S_{01''}$);
V_{FF} = free-flow travel time on segment (mph);
a_n, b_n = nth percentile coefficients for nonprecipitation components (d/c and LHL);
c_n, d_n = nth percentile coefficients for rain and snow components, respectively ($d/c < 0.8$);
$c1_n, c2_n$ = nth percentile coefficients for rain component ($d/c > 0.8$); and
$d1_n, d2_n$ = nth percentile coefficients for snow component ($d/c > 0.8$).

The four primary variables (LHL, d/c, $R_{05''}$, and $S_{01''}$) are discussed above under Model Variables. Table 4.5 shows the TTI prediction model coefficients for $d/c \le 0.8$ and $d/c > 0.8$.

For the $d/c \le 0.8$ models, the four coefficients (a_n, b_n, c_n, d_n) were developed as continuous functions of the TTI percentile (n), allowing prediction of any percentile value (the entire cumulative TTI curve), not just the five percentiles shown in Table 4.5. These coefficient functions are built with subcoefficients, as shown in Equation 4.11. Table 4.6 shows

Table 4.5. TTI Prediction Model Coefficients

N (percentile)	***d/c* ≤ 0.8**				***d/c* > 0.8**					
	a_n	b_n	c_n	d_n	a_n	b_n	$c1_n$	$c2_n$	$d1_n$	$d2_n$
10	0.01400	0.00099	0.00015	0.00037	0.07643	0.00405	1.364	–28.34	0.178	15.55
50	0.07000	0.00495	0.00075	0.00184	0.29097	0.01380	0.966	–6.74	0.345	3.27
80	0.11214	0.00793	0.00120	0.00310	0.52013	0.01544	0.630	6.89	0.233	5.24
95	0.19763	0.01557	0.00197	0.01056	0.63071	0.01219	0.639	5.04	0.286	1.67
99	0.47282	0.04170	0.00300	0.02293	1.13062	0.01242	0.607	5.27	0.341	–0.55

Note: Coefficients for *d/c* ≤ 0.8 are continuous functions of *n*. See text for more description.

Table 4.6. Subcoefficient Values for TTI Prediction Model (d/c < 0.8)

coeff$_n$	**Subcoefficient**			
	w	*x*	*y*	*z*
a_n	0.14	0.504	96	9
b_n	0.0099	0.0481	96	9
c_n	0.00149	0.0197	68	6
d_n	0.00367	0.0248	36	7

the subcoefficient values for the TTI prediction model with $d/c < 0.8$.

$$\text{coeff}_n = wn + xy^{z(n-1)} \tag{4.11}$$

where

coeff$_n$ = one of the four coefficients in the TTI$_n$ formula (a_n, b_n, c_n, d_n);

n = percentile (scaled between 0 and 1.0); and

w, x, y, z = subcoefficients.

Quantifying Design Treatment Effects on Reliability by Using the Cumulative TTI Curve

The preceding section included a detailed explanation of methods to construct a predictive cumulative TTI curve using four primary variables (d/c, LHL, $R_{0.05''}$, and $S_{0.01''}$). Chapter 1 of this report describes how various reliability and delay measures can be extracted from this curve. This section describes how the impacts of highway design treatments can be mapped to the four variables, and how the cumulative TTI curve can be used to evaluate the effectiveness of a design treatment at improving reliability, by comparing TTI curves for the untreated and treated conditions. For various reasons, not all treatments studied in Project L07 are discussed in this section:

- Some treatments do not affect reliability, or reliability variables, in a way that can be meaningfully predicted by the models. For example, ramp closures, such as gates used during flooding events, make a freeway reliable in the sense that it has no congestion—by virtue of its carrying no traffic.
- Some treatments are beyond the scope of the reliability models. For example, improvements to diversion routes may need to be modeled using travel-demand models.

Mapping Treatment Effects to Model Variables

To enable calculation of the reliability effects of highway design treatments, it is necessary to determine how each treatment affects the independent variables in the TTI prediction models. This principle can be represented as shown by Equations 4.12 and 4.13:

Untreated:

$$\text{TTI} = f\{Td/c, \text{ILHL}, \text{WZLHL}, R_{0.05''}, S_{0.01''}\} \tag{4.12}$$

Treated:

$$\text{TTI}^* = f\{d/c^*, \text{ILHL}^*, \text{WZLHL}^*, R^*_{0.05''}, S^*_{0.01''}\} \tag{4.13}$$

where f is a mathematical function (as described under Calculating Demand, starting on page 24), and an asterisk (*) indicates the variable as affected by the treatment (recall that ILHL + WZLHL = LHL).

In this section, treatments are classified by which of these five variables they affect. Most treatments only affect one of the five, although some affect more than one.

Class I: Demand-to-Capacity Ratio

Many design treatments aimed at recurrent congestion can also affect nonrecurrent congestion and reliability, and this effect is captured in the model variable *d/c*.

Case IA: Base Capacity Improvements

Base capacity improvements could include adding a lane or lanes, increasing lane width, adding a shoulder, or increasing shoulder width. Although only the latter two are specifically addressed in the Project L07 research, all these treatments have the same general effect: they increase the *c* term in the denominator of *d/c,* as expressed in Equation 4.14:

$$d/c^* = \frac{d}{c^*} = \frac{d}{rc} \tag{4.14}$$

where

c^* = treated capacity;
c = original capacity;
r = ratio between c^* and c (i.e., c^*/c);
d = demand (here assumed unchanged); and
d/c^* = resulting demand-to-capacity ratio.

For lane additions, $r = N_L^*/N_L$ (the ratio of the number of lanes after treatment implementation to the original number of lanes). For increased lane width, $r = f_{\mathrm{LW}}^*/f_{\mathrm{LW}}$ (the ratio of the treated and untreated HCM lane width adjustment factors for the respective widths). Similarly, shoulder addition or widening is based on f_{LC}, the HCM adjustment factor for lateral clearance.

Case IB: Demand Reductions

Demand-reduction strategies could include the construction of alternate routes, relief of bottlenecks on existing alternate routes, or introduction of high-occupancy vehicle lanes (which have more complex effects beyond pure demand reduction). Case IB strategies affect the numerator of *d/c,* as shown in Equation 4.15:

$$d/c^* = \frac{d^*}{c} = \frac{rd}{c} \tag{4.15}$$

where

d^* = demand after strategy implementation;
d = original demand;
r = ratio between d^* and d (d^*/d);
c = capacity (here assumed unchanged); and
d/c^* = resulting demand-to-capacity ratio.

Class II: Incident Lane Hours Lost

Many of the design treatments studied fall into Class II; that is, they affect ILHL. ILHL can be calculated for various incident types. For each type *i,* the treatment can affect any of three variables (previously defined in Model Variables), as shown by Equation 4.16:

$$\mathrm{ILHL}_i^* = \frac{N_{\mathrm{incidents},i}^* N_{\mathrm{blocked},i}^* T_{\mathrm{incidents},i}^*}{60} \tag{4.16}$$

Class II is further subdivided into six cases, as described in the following subsections. These cases are not necessarily exhaustive, but they cover the relevant nonrecurrent congestion design treatments studied and provide a guide that could be extrapolated to other types of ILHL-reducing treatments. In all the described cases, $N_{\mathrm{blocked},i} = R_{\mathrm{cap},i}N_{\mathrm{lanes}}$ except where noted. See Table 4.3 for appropriate values of $R_{\mathrm{cap},i}$.

Case IIA: Incident Elimination with Unspecified Average Treatable Incident Duration

For treatments that eliminate a fraction (p_i) of incidents of type *i,* only the remaining incidents of that type ($1 - p_i$) contribute to ILHL. For Case IIA, it is assumed that additional information is either unknown or unneeded regarding the duration of the type of incidents for which the treatment will be applied. In this case, only one variable is affected, as shown by Equation 4.17:

$$N_{\mathrm{incidents},i}^* = (1 - p_i) N_{\mathrm{incidents},i} \tag{4.17}$$

The ($1 - p_i$) term is directly related to the concept of a crash modification factor (CMF). Formally introduced to practice through the AASHTO *Highway Safety Manual* (*5*), CMFs can be defined as the ratio of the expected average crash frequency in a treated condition to the expected crash frequency in the untreated condition. Because the frequency is defined as the number of incidents over a specified period, the following logic applies, as shown by Equation 4.18:

$$N_{\mathrm{incidents},i}^* = \mathrm{CMF} \times N_{\mathrm{incidents},i} = (1 - p_i) N_{\mathrm{incidents},i}$$
$$\Rightarrow p_i = 1 - \mathrm{CMF} \tag{4.18}$$

Thus, if the CMF is known for a particular treatment, p_i can be easily calculated for that treatment.

The other two variables remain the same as in the untreated condition (i.e., $N_{\mathrm{blocked},i}^* = N_{\mathrm{blocked},i}$ and $T_{\mathrm{incidents},i}^* = T_{\mathrm{incidents},i}$). Treatments in this category include wildlife–vehicle collision reduction, anti-icing, snow fences, and blowing sand reduction.

Case IIB: Incident Elimination with Specified Average Treatable Incident Duration

As with Case IIA, Case IIB covers treatments that eliminate a portion of incidents. However, in this instance it is assumed that the duration of incidents (of a given type) to which the treatment applies is longer than the overall average duration (for that type). In other words, the treatment is likely to be applied only to incidents that are much more severe than average. For these cases, since the average incident duration ($T_{\mathrm{incidents},i}$) in the base condition is already specified, the analyst must specify $T_{\mathrm{treatable}}$, the average incident duration for those incidents to which the treatment will be applied. Thus, the treated duration (applied only to the incidents that remain) is computed as shown by Equations 4.19 and 4.20:

$$T^*_{\text{incidents},i} = \frac{T_{\text{incidents},i} - p_i T_{\text{treatable}}}{1 - p_i} \tag{4.19}$$

with a notable boundary condition:

$$T_{\text{treatable}} \leq \frac{T_{\text{incidents},i}}{p_i} \tag{4.20}$$

$N^*_{\text{incidents}}$ is calculated as in Case IIA (including the same relationship with CMFs), and N^*_{blocked} remains equal to the untreated condition (i.e., $N^*_{\text{blocked}} = N_{\text{blocked}}$). One example of a treatment falling in this category is the runaway truck ramp.

Case IIC: Response Time Reduction

Certain treatments reduce response time, allowing responders to reach (and therefore clear) certain types of incidents more quickly than in the untreated condition. Unlike incidents in Cases IIA and IIB, a Case IIC treated incident is not eliminated, but its duration is shortened. Therefore, ILHL* for a given incident type i is composed of two terms: one for incidents unaffected by the treatment, and one for incidents affected by the treatment (with a reduced duration T^*_i), as shown by Equations 4.21 and 4.22, respectively:

Unaffected incidents:

$$\text{ILHL}^*_1 = (1 - p_i) N_{\text{incidents},i} N_{\text{blocked},i} T_{\text{incidents},i} / 60 \tag{4.21}$$

Affected incidents:

$$\text{ILHL}^*_2 = p_i N_{\text{incidents},i} N_{\text{blocked},i} T^*_i / 60 \tag{4.22}$$

Equation 4.23 gives the total treated ILHL, which is the sum of these two terms:

$$\text{ILHL}^* = \text{ILHL}^*_1 + \text{ILHL}^*_2 \tag{4.23}$$

One example of a treatment falling in this category is emergency access between interchanges. Some other treatments, such as median crossovers or contraflow lanes, could also be used for these purposes but are not studied in detail.

Case IID: Incident Type Conversion with Unspecified Average Treatable Incident Duration

Case IID includes treatments that essentially transform a portion of incidents, midduration, from one type (i) into another type (k), typically by providing an opportunity for incidents to be shifted from lane-blocking to shoulder-blocking. In these cases, ILHL* is composed of three terms: one for incidents unaffected by the treatment; one for incidents affected by the treatment but before treatment implementation with a duration until conversion T^*_i; and one for incidents affected by the treatment after treatment implementation (conversion to the new treatment type), to which the remaining treatment duration is applied. These three variations are expressed by Equations 4.24 through 4.26:

Unaffected incidents:

$$\text{ILHL}^*_1 = (1 - p_i) N_{\text{incidents},i} N_{\text{blocked},i} T_{\text{incidents},i} / 60 \tag{4.24}$$

Affected incidents, preconversion:

$$\text{ILHL}^*_2 = p_i N_{\text{incidents},i} N_{\text{blocked},i} T^*_i / 60 \tag{4.25}$$

Affected incidents, postconversion:

$$\text{ILHL}^*_3 = p_i N_{\text{incidents},i} N_{\text{blocked},k} (T_{\text{incidents},i} - T^*_i) / 60 \tag{4.26}$$

Equation 4.27 gives the total treated ILHL, which is the sum of these three terms:

$$\text{ILHL}^* = \text{ILHL}^*_1 + \text{ILHL}^*_2 + \text{ILHL}^*_3 \tag{4.27}$$

This formulation assumes that the overall incident duration is the same as in the untreated condition; the latter portion of the duration consists of the second incident type (generally, a nonblocking shoulder incident). Treatments in this category include accessible shoulder, alternating shoulder, crash investigation site, and emergency pulloff.

Case IIE: Incident Type Conversion with Specified Average Treatable Incident Duration

Like Case IID, Case IIE includes treatments that essentially transform a portion of incidents, midduration, from one type (i) into another type (k). However, for this treatment type, crashes are more severe, and it is assumed (as in Case IIB) that the duration of incidents (of a given type) to which the treatment applies is longer than the overall average duration (for that type). As in Case IIB, the analyst must specify $T_{\text{treatable}}$, the average duration of incidents to which the treatment will be applied. As in Case IID, ILHL* is composed of three terms: one for incidents unaffected by the treatment, and two for incidents affected by the treatment (with a duration until conversion T^*_i), as shown by Equations 4.28 to 4.30:

Unaffected incidents:

$$\text{ILHL}^*_1 = (1 - p_i) N_{\text{incidents},i} N_{\text{blocked},i} (T_{\text{incidents},i-p_i} T_{\text{treatable},i}) / [60 \times (1 - p_i)] \tag{4.28}$$

Affected incidents, preconversion:

$$\mathrm{ILHL}_2^* = p_i N_{\mathrm{incidents},i} N_{\mathrm{blocked},i} T_i^*/60 \quad (4.29)$$

Affected incidents, postconversion:

$$\mathrm{ILHL}_3^* = p_i N_{\mathrm{incidents},i} (1 - p_r) N_{\mathrm{blocked},k} \left(T_{\mathrm{treatable},i} - T_i^*\right)/60 \quad (4.30)$$

Equation 4.31 gives the total treated ILHL, which is the sum of these three terms:

$$\mathrm{ILHL}^* = \mathrm{ILHL}_1^* + \mathrm{ILHL}_2^* + \mathrm{ILHL}_3^* \quad (4.31)$$

One example of a treatment in this category is the incident screen.

CASE IIF: INCIDENT DIVERSION

Case IIF includes treatments that, when deployed during an incident, allow vehicles upstream of the input to detour via temporary new capacity (either by leaving the mainline or using a shoulder). None of the prediction model variables (as strictly defined) directly addresses the effects of this type of improvement. The two model variables with the greatest potential for addressing incident diversion, ILHL and *d/c*, have the following challenges:

- ILHL is based on incident duration. Diverting vehicles does not shorten incident duration as defined in Case IIC; diversion theoretically has no effect on the time to clear an incident, although it can have a profound effect on the time until normal flow is recovered.
- *d/c* is a measure characterizing the general level of saturation of a facility. It was not designed to emulate the effects of incidents. One might be tempted to use it as a proxy since demand is being diverted and additional capacity is being provided, but rare diversion events would not typically affect annual demand or capacity enough to significantly affect the cumulative TTI curve.

Even though neither of these measures is satisfying, based on tests and theoretical explorations, the Project L07 research team concluded that ILHL was better suited to account for the effects of diversion-related treatments.

Two important parameters need to be defined in conjunction with the analysis of diversion-related treatments:

- The capacity, or throughput, of the diversion treatment itself is termed c_{div}. For example, a gravel crossover may be able to process fewer vehicles per hour than a paved crossover.
- The typical duration of an incident for which the crossover would be used is termed $T_{\mathrm{treatable}}$. As with Cases IIB and IIE, it is assumed that incidents for which this treatment would be deployed have a longer-than-average duration. $T_{\mathrm{treatable}}$ is used in a different way with Case IIF than it is used in Cases IIB and IIE, as discussed below.

For Case IIF, the reduction in LHL is treated as "lane hours gained," as shown by Equations 4.32 and 4.33:

$$\Delta\mathrm{ILHL} = \frac{p_i N_{\mathrm{incidents},i} N_{\mathrm{lanes}} c_{\mathrm{div}} T_{\mathrm{treatable}}}{C} \quad (4.32)$$

$$\mathrm{ILHL}^* = \mathrm{ILHL} + \Delta\mathrm{ILHL} \quad (4.33)$$

Therefore, unlike other cases, ILHL is not made up of three terms, although ΔILHL is made up of three terms analogous to the typical ILHL calculation:

- $p_i N_{\mathrm{incidents},i}$ represents the number of treated incidents.
- $N_{\mathrm{lanes}} c_{\mathrm{div}}/C$ (or $[c_{\mathrm{div}}/C] \times N_{\mathrm{lanes}}$) represents the equivalent number of lanes "unblocked" (c_{div} is in units of vehicles per year, and C is in units of vehicles per lane hour).
- $T_{\mathrm{treatable}}$ represents duration. Although each time-slice covers 1 h of the day, default values of $T_{\mathrm{treatable}}$ are often greater than 1 h. For Case IIF (not unlike Cases IIB and IIE), this duration accrues to a single time-slice, even when longer than 60 min. This simplification yields some lane hour savings accounted for during the "wrong hour," but accumulates correctly when all 24 h of the day are considered.

Treatments in this class include emergency crossovers, controlled or gated turnarounds, drivable shoulders, and movable cable median barriers. Table 4.7 summarizes the ILHL equation terms for each of the six cases and their subcases.

CLASS III: WORK-ZONE LANE HOURS LOST

For short-term work zones, three variables can affect the calculation of WZLHL in the treated condition: the per lane capacity of the work zone, the number of lanes available through the work zone, or the number of days the work zone is active. Short-term WZLHL is calculated as shown by Equation 4.34:

$$\mathrm{WZLHL}^* = \left(1 - \frac{c_{\mathrm{WZ}} N^*_{\mathrm{lanes,WZ}}}{c N^*_{\mathrm{lanes}}}\right) N^*_{\mathrm{days}} \quad (4.34)$$

Therefore, if a treatment affects one of these variables, this formula can be used in the TTI prediction models.

CLASS IV: $R_{0.05''}$

Design treatments do not affect the variable $R_{0.05''}$ because they cannot influence the amount of rain that falls. However,

Table 4.7. Terms in Treated ILHL Equations: Six Class II Cases

Case[a]	$N^*_{incidents,i}$	$N^*_{blocked,i}$	$T^*_{incidents,i}$
IIA: Incident elimination with unspecified average treatable incident duration	$(1 - p_i)N_{inc,i}$	$(1 - R_{cap,i})N_{lanes}$	$T_{inc,i}$
IIB: Incident elimination with specified average treatable incident duration	$(1 - p_i)N_{inc,i}$	$(1 - R_{cap,i})N_{lanes}$	$(T_{inc,i} - p_i T_{treatable})/(1 - p_i)$
IIC: Response time reduction	unaffected: $(1 - p_i)N_{inc,i}$	$(1 - R_{cap,i})N_{lanes}$	$T_{inc,i}$
	affected: $p_i N_{incidents,i}$	$(1 - R_{cap,k})N_{lanes}$	T^*_i
IID: Incident type conversion with unspecified average treatable incident duration, passive treatment	unaffected: $(1 - p_i)N_{inc,i}$	$(1 - R_{cap,i})N_{lanes}$	$T_{inc,i}$
	affected, preconversion: $p_i N_{inc,i}$	$(1 - R_{cap,i})N_{lanes}$	T^*_i
	affected, postconversion: $p_i N_{inc,i}$	$(1 - R_{cap,k})N_{lanes}$	$T_{inc,i} - T^*_i$
IIE: Incident type conversion with specified average treatable incident duration, active treatment	unaffected: $(1 - p_i)N_{inc,i}$	$(1 - R_{cap,i})N_{lanes}$	–
	affected, preconversion: $p_i N_{inc,i}$	$(1 - R_{cap,i})N_{lanes}$	T^*_i
	affected, postconversion: $p_i N_{inc,i}$	$(1 - R_{cap,i})(1 - p_r)N_{lanes}$	$T_{treatable} - T^*_i$
IIF: Incident diversion	$ILHL^* = ILHL + \frac{p_i N_{incidents,i} N_{lanes} c_{div} T_{treatable}}{C}$		

[a] For Cases IIA through IIE, $ILHL^* = N^*_{incidents,i} \times N^*_{blocked,i} \times T^*_{incidents,i}$. For Case IIF, ILHL* is as shown in the table.

$R_{0.05''}$ is an important variable in the model because it helps describe the base conditions.

Class V: $S_{0.01''}$

Just as design treatments do not affect the variable $R_{0.05''}$, they do not affect the variable $S_{0.01''}$ because they cannot influence snowfall. However, $S_{0.01''}$ is an important variable in the model because it helps describe the base conditions. Although the amount of snow that falls cannot be influenced by design treatments, treatments like snow fences may reduce snow accumulation on the roadway and improve visibility, thereby reducing the number of snow-related crashes and, therefore, ILHL.

Table 4.8 provides suggested default coefficient values for treatment mapping to TTI predictions models.

Calculating Operational Effectiveness: Overview

As described in the previous section, each treatment changes reliability by modifying the value of either LHL or *d/c*. For each hour of the day, untreated and treated TTI curves can be generated for a particular freeway segment and placed on the same graph. Thus, the key step in quantifying the effect of design treatments on reliability is to estimate TTI distribution curves, like those shown in Figure 4.4. The area between the untreated and treated TTI curves is proportional to the overall delay reduction resulting from the treatment. Although this report focuses on a specific set of treatments to address nonrecurrent congestion, this approach can be applied to any treatment or operational strategy that can be mapped to at least one of the four variables in the TTI prediction models.

Figure 4.5, presented earlier as Figure 4.1, is re–presented here to illustrate the process that leads to the final calculation of operational benefits.

Many operational measures were introduced in Chapter 1, and the changes in all of them can be computed based on the computed cumulative TTI curves. However, in preparation for calculation of economic benefits (see Chapter 6), the two most important measures are the lateness index (LI) and the standard deviation. Calculation of changes in these measures is discussed in the following two sections. Calculation of other measures, such as the semivariance and various indices, are discussed under Background at the beginning of Chapter 1.

The TTI prediction models have a feature that is important to note in reliability calculations: there is no smooth transition between the $d/c \leq 0.8$ and $d/c > 0.8$ models. This could cause an overestimation of operational benefits if a treatment causes a d/c above 0.8 to decrease below 0.8. Therefore, in all cases, it is recommended that the model used for the untreated condition (with respect to d/c) should be used for the treated conditions, even if the treatment causes d/c to cross the 0.8 boundary.

Change in Lateness Index

As described in Chapter 1, the unitless area between the cumulative TTI curve and the vertical line at TTI = 1.0

Table 4.8. Suggested Default Coefficients or Terms for Treatment Mapping to TTI Prediction Models for Class II (Incident Lane Hours Lost)

Treatment	Case	Portion of Incidents Using or Affected by Treatment, p_i						Ratio of Applicable Duration to $T_{incident}$		
		Crashes[a]			Noncrashes[a]					
		PDO	Min	Maj/Fat	NLB	LB	Other	PDO	Min	Maj/Fat
Anti-icing systems	IIA	*0.10*	*0.10*	*0.10*	na	na	na			
Blowing sand	IIA	*0*	*0*	*0*	na	na	na			
Extra-high median barrier	IIA	Apply to opposite-direction incidents								
Snow fence	IIA	*0.10*	*0.10*	*0.10*	na	na	na			
Wildlife collision reduction	IIA	[b]	[b]	[b]	na	na	[b]			
Runaway truck ramp	IIB	*0.001*	*0.001*	*0.001*	na	na	na	4	4	4

Treatment	Case	Portion of Incidents Using or Affected by Treatment, p_i						Incident Duration with Treatment, T^* (min)					
		Crashes[a]			Noncrashes[a]			Crashes			Noncrashes		
		PDO	Min	Maj/Fat	NLB	LB	Other	PDO	Min	Maj/Fat	NLB	LB	Other
Emergency access between interchanges	IIC	*0.05*	*0.10*	*0.20*	na	na	na	5	5	5	na	na	na
Accessible shoulder	IID	*0.50*	*0.30*	*0.10*	na	*0.60*	*0.25*	25	35	45	na	20	20
Alternating shoulder	IID	*0.35*	*0.25*	*0.05*	na	*0.50*	*0.20*	25	35	45	15	20	20
Crash investigation site	IID	*0.40*	*0.20*	*0*	*0.20*	*0.40*	*0.10*	25	35	45	15	20	20
Emergency pulloff	IID	*0.40*	*0.20*	*0*	na	*0.15*	*0.10*	25	35	45	15	20	20

Treatment	Case	PDO	Min	Maj/Fat	Deployment Time[a] (min)	Ratio of Applicable Duration to Average Incident Length			Lost Capacity Restored (%), $p_{r,i}$		
						PDO	Min	Maj/Fat	PDO	Min	Maj/Fat
Incident screens	IIE	*0*	*0.05*	*0.10*	*20*	2	2	2	10	10	10

Treatment	Case	PDO	Min	Maj/Fat	NLB	LB	Other	Applicable Duration (h)			v/c Threshold of Application		
								PDO	Min	Maj/Fat	PDO	Min	Maj/Fat
Emergency crossovers	IIF	*0*	*0.01*	*0.05*	na	na	na	1.0	1.5	2.0	1.0	1.0	1.0
Controlled/gated turnarounds	IIF	*0*	*0.01*	*0.05*	na	na	na	1.0	1.5	2.0	1.0	1.0	1.0
Drivable shoulder	IIF	*0.05*	*0.15*	*0.25*	na	*0.05*	*0.05*	1.0	1.5	2.0	1.0	1.0	1.0
Movable cable median barrier	IIF	*0*	*0.01*	*0.05*	na	na	na	1.0	1.5	2.0	1.0	1.0	1.0

[a] Italicized values are user-modifiable in the Analysis Tool.
[b] There are many varieties of wildlife collisions, each with its own set of potential treatments and effects.
Note: Min = minor injury; Maj/Fat = major injury or fatality; NLB = non-lane-blocking; LB = lane-blocking; na = not applicable.

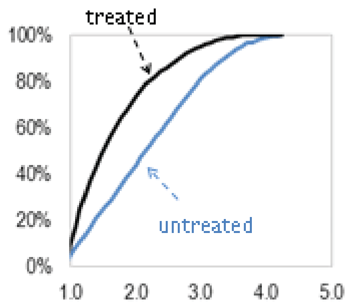


Figure 4.4. Comparison of treated and untreated TTI curves.

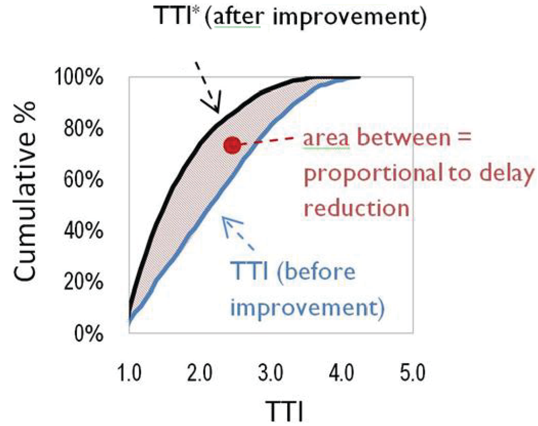


Figure 4.6. Change in lateness index.

(i.e., the LI) is a measure similar to the mean of the TTI distribution. The difference between untreated and treated LIs is equal to the area between the two curves (see Figure 4.6) and is proportional to the overall delay savings resulting from the treatment, because one can think of the reduced travel time at each TTI percentile as being applicable to the vehicles represented in that percentile. This unitless area can be multiplied by the vehicle volume for the time-slice and the free-flow travel time for the segment, resulting in a value that represents vehicle hours of delay reduced by implementing the treatment.

To calculate delay, TTI is converted to an actual segment travel time. This can be accomplished with the following procedure. The free-flow travel time (TT_{FF}) is defined as the segment length (L) divided by the free-flow travel speed (S_{FF}), as shown by Equation 4.35:

$$TT_{FF} = \frac{L}{S_{FF}} \tag{4.35}$$

Percentiles of the TTI curve can be determined from the TTI prediction models described above under Prediction Models, as shown in Equation 4.36:

$$TTI_n = \begin{cases} TTI_{NP,n} \times e^{(c_n R_{0.05''} + d_n S_{01''})} & \text{for } d/c \le 0.8 \\[2ex] \dfrac{TTI_{NP,n}}{N_{days}} \times \left[N_{NP} + V_{FF} \left(\dfrac{R_{0.05''}}{c1_n V_{FF} + c2_n TTI_{NP,n}} + \dfrac{S_{01''}}{d1_n V_{FF} + d2_n TTI_{NP,n}} \right) \right] & \\ & \text{for } d/c > 0.8 \end{cases} \tag{4.36}$$

The actual travel time (TT) corresponding to any given value of TTI is given by Equation 4.37:

$$TT = (TTI)(TT_{FF}) = TTI\left(\frac{L}{S_{FF}}\right) \tag{4.37}$$

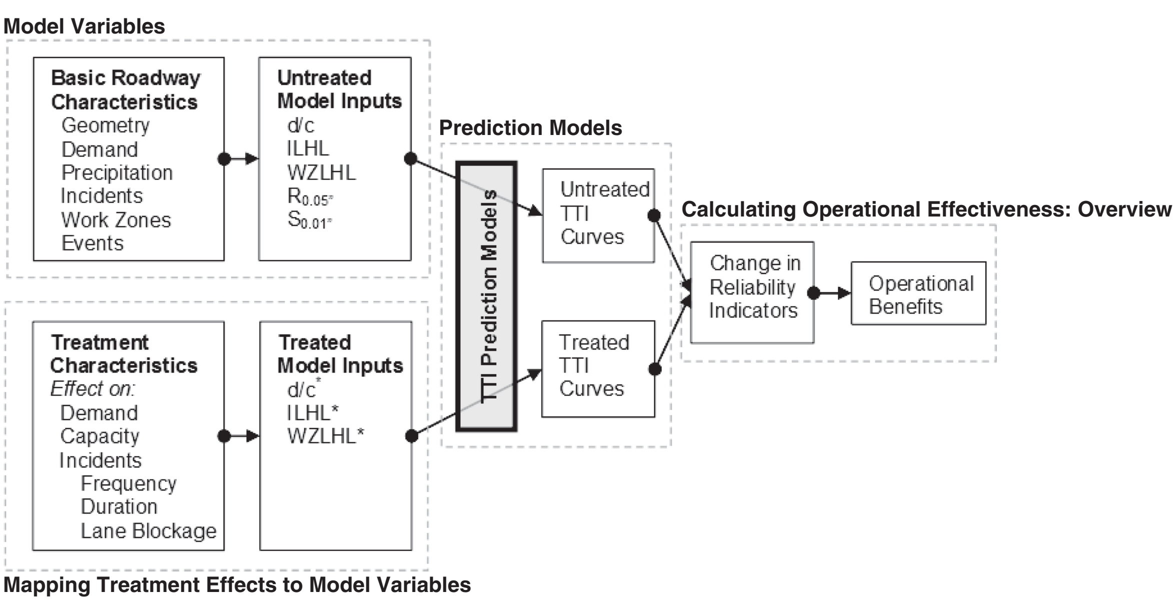


Figure 4.5. Calculation of operational effectiveness.

Therefore, the travel time savings (ΔTT_n)—and by implication, delay reduction—at a given percentile n can be calculated as shown by Equation 4.38:

$$\Delta TT_n = TT_n - TT_n^* = \left(TTI_n - TTI_n^*\right)\frac{L}{S_{FF}} \tag{4.38}$$

where

TT_n = travel time (h) for percentile n of the cumulative travel time distribution (TT-CDF) in the untreated condition;

TT_n^* = travel time (h) for percentile n of the TT-CDF in the treated condition;

TTI_n = TTI for percentile n of the cumulative TTI distribution (TTI-CDF) in the untreated condition; and

TTI_n^* = TTI for percentile n of the TTI-CDF in the treated condition.

If the treated and untreated TTI curves shown in Figure 4.6 were continuous functions (and note that the TTI prediction function for $d/c \le 0.8$ does predict continuous distributions), the total vehicle hours of delay (or change in LI) for the entire time-slice could be calculated as shown by Equation 4.39:

$$\Delta LI_k = N_d V L S_{ff} \int_{i=0}^{100\%} \left(TTI_i - TTI_i^*\right) di = N_d V L S_{ff} \int_{i=0}^{100\%} \Delta TTI_i \, di \tag{4.39}$$

where

ΔLI_k = traffic operational delay reduction due to design treatment during time-slice k (change in LI);

N_d = number of days in the time-slice (generally assumed as 250 nonholiday weekdays); and

V = hourly vehicular volume during the time-slice.

All other variables are as described previously.

However, as the TTI prediction functions for $d/c > 0.8$ predict five discrete percentiles of the cumulative TTI distribution, rather than a continuous curve, the area between the curves must be approximated by trapezoids, as illustrated in Figure 4.7 (summing A1, A2, A3, and A4). Given that the area of a trapezoid is one-half the sum of the two parallel sides multiplied by the distance between them, and simplifying terms, the area can be approximated as expressed by Equation 4.40:

$$\Delta LI_k \approx N_d V L S_{ff} \left(0.200 \Delta TTI_{10\%} + 0.350 \Delta TTI_{50\%} + 0.225 \Delta TTI_{80\%} + 0.095 \Delta TTI_{95\%} + 0.020 \Delta TTI_{99\%}\right) \tag{4.40}$$

This sum omits the area of the small tails at either end of the distribution that are considered negligible for the purposes of this analysis.

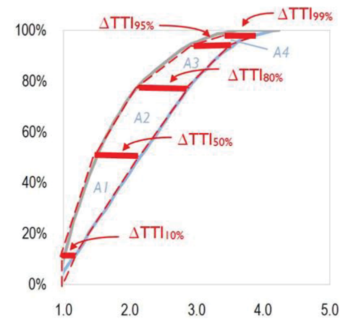


Figure 4.7. Estimating delay by quantifying the area between treated and untreated TTI curves.

Change in Variance

Project L07 also focused on the reduction in the variance or standard deviation of travel time as a reliability measure, because that measure has an economic interpretation documented in the literature. Therefore, the computation of the standard deviation of travel time is the focus of the following discussion. However, the state of knowledge about reliability and its economic value is rapidly evolving.

If the entire distribution were known, the variance would be computed as indicated in Chapter 1 and expressed by Equation 4.41:

$$\sigma = \int_{i=0}^{100\%} \left(TTI_i - TTI_{mean}\right)^2 di \tag{4.41}$$

Because the TTI prediction functions do not provide a continuous distribution, the area under the five-point "curve" in Figure 4.8 is a reasonable approximation for the variance (σ).

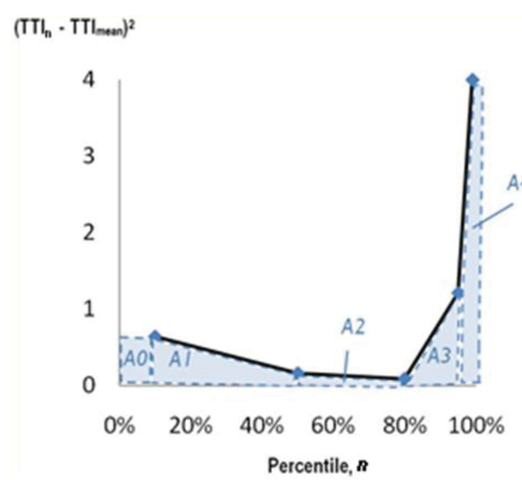


Figure 4.8. Graphical presentation of procedure for approximating the variance of the TTI distribution.

Because the x-axis is expressed in percentages, no normalizing constant is needed. Using calculations for the trapezoidal Areas A1 through A4 and simplifying expressions yields Equation 4.42:

$$\sigma \approx S_{FF}\left(0.300\Delta_{10\%} + 0.350\Delta_{50\%} + 0.225\Delta_{80\%} + 0.095\Delta_{95\%} + 0.020\Delta_{99\%}\right) \tag{4.42}$$

where Δ_n equals $(\mathrm{TTI}_n - \mathrm{TTI}_{mean})^2$.

Similar approximations can be made in the case of semivariance (σ_r), as shown by Equation 4.43:

$$\sigma_r = \int_{i=0}^{100\%} (\mathrm{TTI}_i - 1)^2\, di \tag{4.43}$$

However, the standard deviation is the measure suggested by the literature for calculating the economic value of reliability.

Change in Other Reliability Measures

The cumulative TTI curves for the treated and untreated conditions can also be used to derive any of the remaining reliability indicators presented in Chapter 1, most of which are indices. In general, the difference between the treated and untreated indices can be computed for each hour of the day, but no 24-h summary measures have yet been developed for any of these indicators. It is illuminating to plot each of these indices as they vary by time of day, not only for treated and untreated conditions, but for the difference between the two.

CHAPTER 5

Safety Assessment of Design Treatments

The objective of the safety analysis was to estimate, in quantitative terms, the safety effectiveness for each treatment of interest. Design treatments to reduce nonrecurrent congestion have two potential effects on safety for the highway facilities on which they are implemented:

- Design treatments may have a direct effect on crash frequency or severity if they affect the speeds or lateral positions of vehicles. Effects on crash frequency may result from treatments that change lane width, shoulder width, or other geometric features related to the base capacity of the facility, as indicated by *Highway Capacity Manual* (HCM) procedures (*2*). Crash severity may be affected by design treatments that change the roadside design of the facility.
- Design treatments may have an indirect effect on crash frequency if they reduce congestion on the highway facility. The relationship between congestion and crash frequency is documented in this section.

Although the direct effects of design treatments on crash frequency for freeways have not been fully documented, they have recently been investigated in NCHRP Project 17-45 for inclusion in the *Highway Safety Manual* (*5*). This research has documented the effect on safety of changing the inside and outside shoulder width on freeways. The direct effect of design treatments on roadside crash severity can be estimated with the *Roadside Safety Analysis Program* (*6*). The relationship between congestion and safety has been determined in Project L07 and included in the assessment of design treatments.

Direct Effects of Design Treatments on Safety

A new safety prediction methodology for freeways has been developed in NCHRP Project 17-45 (*4*). This methodology is currently in the approval process for inclusion in the *Highway Safety Manual* (*5*). The only variables in the safety prediction methodology that appear to relate directly to the assessment of design treatments are outside shoulder width and inside shoulder width.

The effect of outside shoulder width on safety on a tangent roadway section is represented by the following crash modification factor (CMF), as given by Equation 5.1:

$$\text{CMF} = \exp\left(a\left[W_{os} - 10\right]\right) \tag{5.1}$$

where

W_{os} = outside shoulder width on freeway section (ft; range, 4 to 14 ft); and

a = regression coefficient (–0.0647 for fatal-and-injury [FI] crashes and 0.0000 for property-damage-only [PDO] crashes).

The percentage change in crashes resulting from a change in outside shoulder width can be determined from the CMFs in Table 5.1 and as shown by Equation 5.2:

$$\text{Percentage change in crashes} = (\text{CMF} - 1) \times 100 \tag{5.2}$$

Thus, a CMF of 1.03 corresponds to a 3% increase in crash frequency, and a CMF of 0.97 corresponds to a 3% decrease in crash frequency.

The effect of inside shoulder width on safety is given by Equation 5.3:

$$\text{CMF} = \exp\left(a\left[W_{is} - 6\right]\right) \tag{5.3}$$

where

W_{is} = inside shoulder width on freeway section (ft; range, 2 to 12 ft); and

a = regression coefficient (–0.0172 for FI crashes and –0.0153 for PDO crashes).

Table 5.2 shows CMFs for the effect on safety of changing inside shoulder width.

Table 5.1. CMFs for Changing Outside Shoulder Width on Freeways (4)

Outside Shoulder Width (ft) (before)	Outside Shoulder Width (ft) (after)					
	4	6	8	10	12	14
	Fatal-and-Injury Crashes					
4	1.00	0.88	0.77	0.68	0.60	0.52
6	1.14	1.00	0.88	0.77	0.68	0.60
8	1.30	1.14	1.00	0.88	0.77	0.68
10	1.47	1.30	1.14	1.00	0.88	0.77
12	1.68	1.47	1.30	1.14	1.00	0.88
14	1.91	1.68	1.47	1.30	1.14	1.00
	Property-Damage-Only Crashes					
4	1.00	1.00	1.00	1.00	1.00	1.00
6	1.00	1.00	1.00	1.00	1.00	1.00
8	1.00	1.00	1.00	1.00	1.00	1.00
10	1.00	1.00	1.00	1.00	1.00	1.00
12	1.00	1.00	1.00	1.00	1.00	1.00
14	1.00	1.00	1.00	1.00	1.00	1.00

Table 5.2. CMFs for Changing Inside Shoulder Width on Freeways (4)

Inside Shoulder Width (ft) (before)	Inside Shoulder Width (ft) (after)					
	2	4	6	8	10	12
	Fatal-and-Injury Crashes					
2	1.00	0.97	0.93	0.90	0.87	0.84
4	1.03	1.00	0.97	0.93	0.90	0.87
6	1.07	1.03	1.00	0.97	0.93	0.90
8	1.11	1.07	1.03	1.00	0.97	0.93
10	1.15	1.11	1.07	1.03	1.00	0.97
12	1.19	1.15	1.11	1.07	1.03	1.00
	Property-Damage-Only Crashes					
2	1.00	0.97	0.94	0.91	0.88	0.86
4	1.03	1.00	0.97	0.94	0.91	0.88
6	1.06	1.03	1.00	0.97	0.94	0.91
8	1.10	1.06	1.03	1.00	0.97	0.94
10	1.13	1.10	1.06	1.03	1.00	0.97
12	1.17	1.13	1.10	1.06	1.03	1.00

Tables 5.1 and 5.2 can be used by users of the Analysis Tool to determine the direct effects of changing outside or inside shoulder width on safety. The design treatments to which these effects potentially apply are as follows:

- Accessible shoulder
  - Inside shoulder width
  - Outside shoulder width
- Drivable shoulder
  - Inside shoulder width
  - Outside shoulder width
- Alternating shoulder
  - Inside shoulder width
  - Outside shoulder width

Development of Congestion–Safety Relationship

The reduction of congestion through application of design treatments or intelligent transportation system improvements has been widely thought to have a positive effect on safety, but this relationship has not been well quantified in previous research. Congestion may result in stalled or slowed traffic, and the situation in which high-speed vehicles approach the rear of an unexpected traffic queue clearly presents a substantial risk of collision. There is also a clear potential for collision within queues of stop-and-go traffic. The frequency of both of these conditions can be ameliorated by treatments to reduce nonrecurrent congestion. However, collision severity is clearly a function of speed, so the lower speeds on roadways during congested periods may reduce overall collision severity. This trade-off between crash frequency and severity in congested versus uncongested conditions had never been satisfactorily quantified. Research on this issue for freeway facilities has been conducted by Zhou and Sisiopiku (*7*) and Hall and Pendleton (*8*). In particular, Zhou and Sisiopiku suggest that different crash types respond in different ways to volume-to-capacity ratios based on hourly volumes. The research results presented below illustrate why a difference between crash types appears reasonable.

To determine a relationship between safety and congestion for use in evaluating design treatments, relationships between crash rates and level of service (LOS) were developed that were based on 3 years (2005 to 2007) of data obtained from freeways in two metropolitan areas: Seattle, Washington, and Minneapolis–St. Paul, Minnesota. The selection of the two metropolitan areas was based on the availability of relevant data; the sites in Minneapolis–St. Paul included two to five directional lanes of travel, and those in Seattle included two to four directional lanes of travel. Each station for which traffic volume and speed data were available included detectors in each lane across one direction of travel on a freeway. For

Table 5.3. Site Distribution Characteristics for Directional Freeway Segments in Seattle and Minneapolis–St. Paul

Metropolitan Area	No. of Directional Lanes[a]	No. of Sites	Length (mi)	No. of 15-min Records[b]
Seattle, Washington	2	66	93.8	6,937,920
	3	56	81.9	5,886,720
	4	23	24.1	2,417,760
	All lanes	**145**	**199.8**	**15,242,400**
Minneapolis–St. Paul, Minnesota	2	151	146.0	15,780,000
	3	185	184.8	19,412,448
	4	73	67.6	7,673,760
	5	10	11.7	1,051,200
	All lanes	**419**	**410.1**	**43,917,408**

[a] Not including high-occupancy vehicle lanes.
[b] Includes records with missing volume or speed values.

analysis purposes, the freeway system was divided into directional segments, usually extending from one interchange to the next. The sections were selected so that a given detector would be representative of the traffic conditions for all crashes within that section. The most appropriate station was selected for each directional segment; whenever possible, a station near the center of a segment was selected. Table 5.3, which summarizes the available site data, shows that there were 145 roadway sections representing 200 mi of directional freeway segments in Seattle and 419 roadway sections representing 410 mi of directional freeway segments in Minneapolis–St. Paul.

Database Development

The original detector data collected at each station on the freeways consisted of 5-min volume and average speed data for each travel lane; speeds or volumes were missing for some 5-min intervals on one or more lanes. Most missing data were attributed to detector malfunctions. As no set of loop detectors will function across all freeway lanes all the time, some missing volume and speed data are inevitable. A detector that malfunctions is usually out of service for a substantial time period; however, there is no reason to believe that missing data due to a malfunctioning detector leads to a bias in the remaining data set. Missing traffic volume data could not be estimated and were treated as missing. Missing speed data were estimated as the average of the speeds for the adjacent lanes on both sides of the missing lane, as long as the two speeds being averaged were within 5 mph of one another. Speed data were estimated only when volume data were available. If the difference between the speeds in the lanes adjacent to the missing lane was greater than 5 mph, traffic conditions were considered to be too nonhomogeneous to estimate the missing speed. The percentage of time periods with missing data was approximately 19% of the 3-year study period for Seattle and 16% for Minneapolis–St. Paul. In addition, a decision was reached to exclude from the study all data in the Minneapolis–St. Paul area after the I-35W bridge collapse on August 1, 2007, which resulted in unusual flow conditions. Although this period might have been interesting (because volumes changed dramatically on many freeway segments), the changed driving conditions were new to many drivers, and the Minnesota Department of Transportation (DOT) made many modifications to specific roadways to increase base capacity; complete documentation of all these changes and their geometrics are not readily available.

Flow rates in vehicles per hour per lane were computed from the data for each station both for each lane and for all lanes combined based on the available 5-min volume data. These flow rates included some large fluctuations. The speed and volume data were aggregated into 15-min intervals, which provided much more stable data. Once processed, the volume and speed data were used to determine the LOS for each 15-min interval (discussed later in this section).

Crash data for each directional freeway segment were compiled for the same 15-min periods as the traffic volume and speed detector data on the basis of the reported crash date and time. The crash data, which were obtained through the Highway Safety Information System, included all mainline freeway crashes that occurred within the limits of each roadway section of interest during the study period. Crash severity levels considered in the evaluation were as follows:

- Total crashes (i.e., all crash severity levels combined)
- FI crashes
- PDO crashes

Table 5.4. Crash Distribution by Collision Type and Crash Severity for Freeway Sections in Minneapolis–St. Paul and Seattle

Collision Type	No. (%) of Crashes by Crash Severity									
	Fatal		A Injury		B Injury		C Injury		PDO	
Minneapolis–St. Paul										
Single vehicle	5	(35.7)	12	(37.5)	127	(41.0)	297	(21.7)	939	(20.7)
Multiple vehicle	9	(64.3)	20	(62.5)	183	(59.0)	1,070	(78.3)	3,594	(79.3)
All	14	(100)	32	(100)	310	(100)	1,367	(100)	4,533	(100)
Seattle										
Single vehicle	17	(68.0)	32	(36.0)	214	(31.8)	639	(14.6)	1,449	(15.0)
Multiple vehicle	8	(32.0)	57	(64.0)	459	(68.2)	3,745	(85.4)	8,220	(85.0)
All	25	(100)	89	(100)	673	(100)	4,384	(100)	9,669	(100)

Table 5.4 summarizes the crash data (number and percentage) by collision type and severity separately for Seattle and Minneapolis–St. Paul over the 3-year period.

Level of Service Calculations

LOS was computed for each 15-min record by using the operational analysis procedure presented in HCM, Chapter 23 (*2*). Components in the LOS calculations included directional volume, directional speed, flow rates, traffic mix adjustment factor to determine flow rates in passenger cars per hour per lane (i.e., heavy-vehicle adjustment factor), and traffic density. Truck percentages for each roadway section were obtained from maps and other data published by the state DOT or the relevant metropolitan planning organization.

The operational measure used to define LOS for freeways is the traffic density in passenger cars per hour per mile. The traffic density for a 15-min period was computed from the available speed and volume data, as shown by Equation 5.4:

$$D_{15} = \frac{4\,V_{15}\,f_{HV}}{S_{15}} \tag{5.4}$$

where
- D_{15} = traffic density for a 15-min period;
- V_{15} = traffic volume (number of vehicles) for the 15-min period summed across all lanes;
- f_{HV} = heavy-vehicle adjustment factor from HCM, Equation 23-3 (assuming site-specific truck percentage and zero recreational vehicles); and
- S_{15} = average spot speed across all lanes (weighted by lane volumes) (mph).

Equation 5.4 does not include the peak hour factor; that is, D_{15} is based on the actual 15-min volume, not the highest 15-min volume during a particular hour, as is commonly used in HCM procedures.

As specified in the HCM, six LOS categories are assigned by traffic density ranges as follows:

- LOS A: 0 to 11 passenger cars per mile per lane (pc/mi/lane)
- LOS B: 11 to 18 pc/mi/lane
- LOS C: 18 to 26 pc/mi/lane
- LOS D: 26 to 35 pc/mi/lane
- LOS E: 35 to 45 pc/mi/lane
- LOS F: 45+ pc/mi/lane

As the LOS categories are quite broad, a more refined LOS categorization was used to better capture the relationship between density and crash rates. The 18 LOS categories selected are shown in Table 5.5.

Table 5.5. LOS Categories Used in the Study

LOS	Traffic Density Range (pc/mi/lane)	LOS	Traffic Density Range (pc/mi/lane)
A+	0 to 3	D+	26 to 29
A	3 to 7	D	29 to 32
A–	7 to 11	D–	32 to 35
B+	11 to 13	E+	35 to 38
B	13 to 15	E	38 to 41
B–	15 to 18	E–	41 to 45
C+	18 to 20	F+	45 to 50
C	20 to 23	F	50 to 55
C–	23 to 26	F–	55+

Development of LOS–Crash Rate Relationships

Based on the 15-min crash rate and traffic density data, average crash rates (expressed in crashes per million vehicle miles of travel [MVMT]) were calculated within each of the 18 LOS categories, separately for each severity level and each metropolitan area. Similarly, average densities were calculated within each of the 18 LOS categories for each metropolitan area. The resulting pairs of data points are plotted by severity level and metropolitan area in Figures 5.1 and 5.2.

Figure 5.1 shows the variation of crash rate per MVMT with traffic density for freeway sections in the Seattle metropolitan area. Each point represents the crash rate for all 15-min periods of the 3-year period that fell in a particular

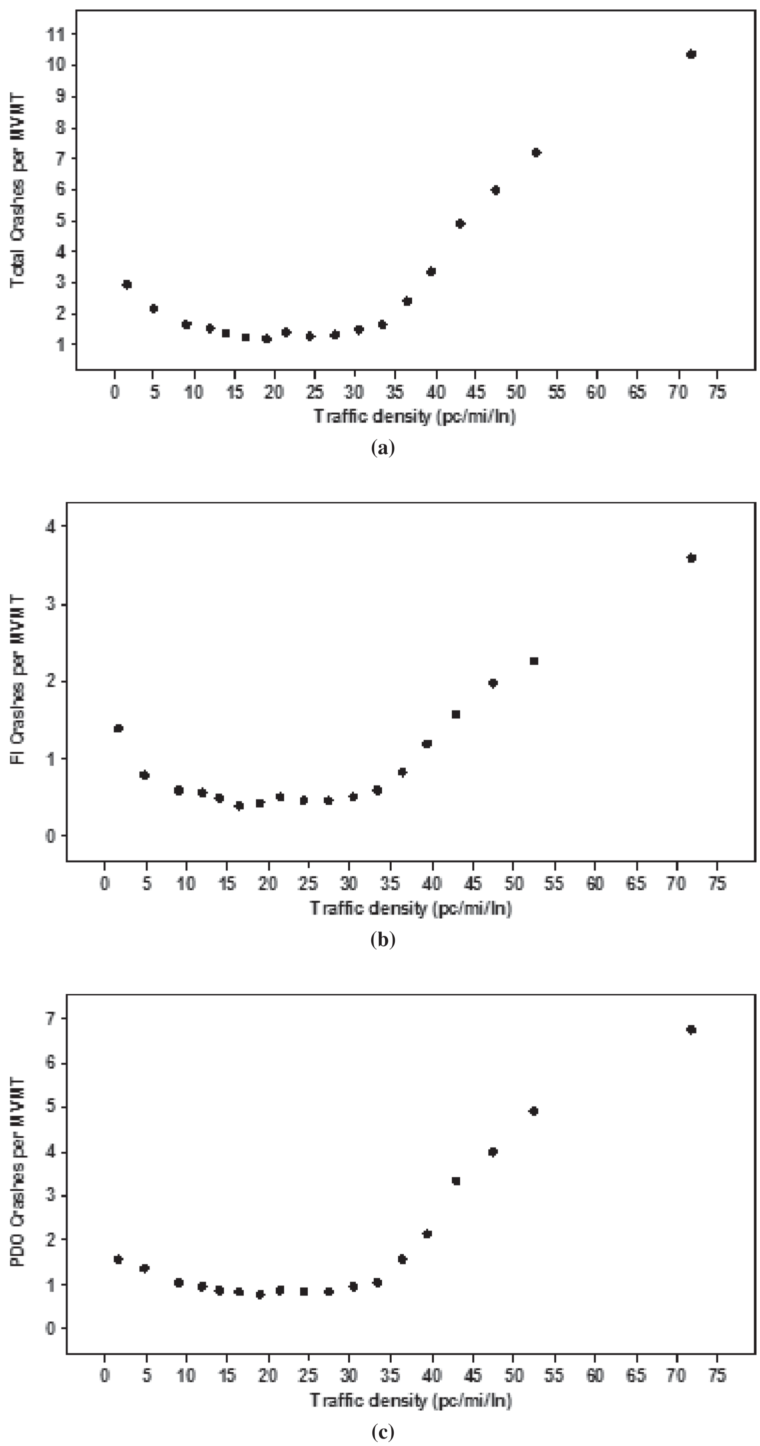


Figure 5.1. Freeway traffic density versus (a) total, (b) FI, and (c) PDO crash rates in the Seattle area.

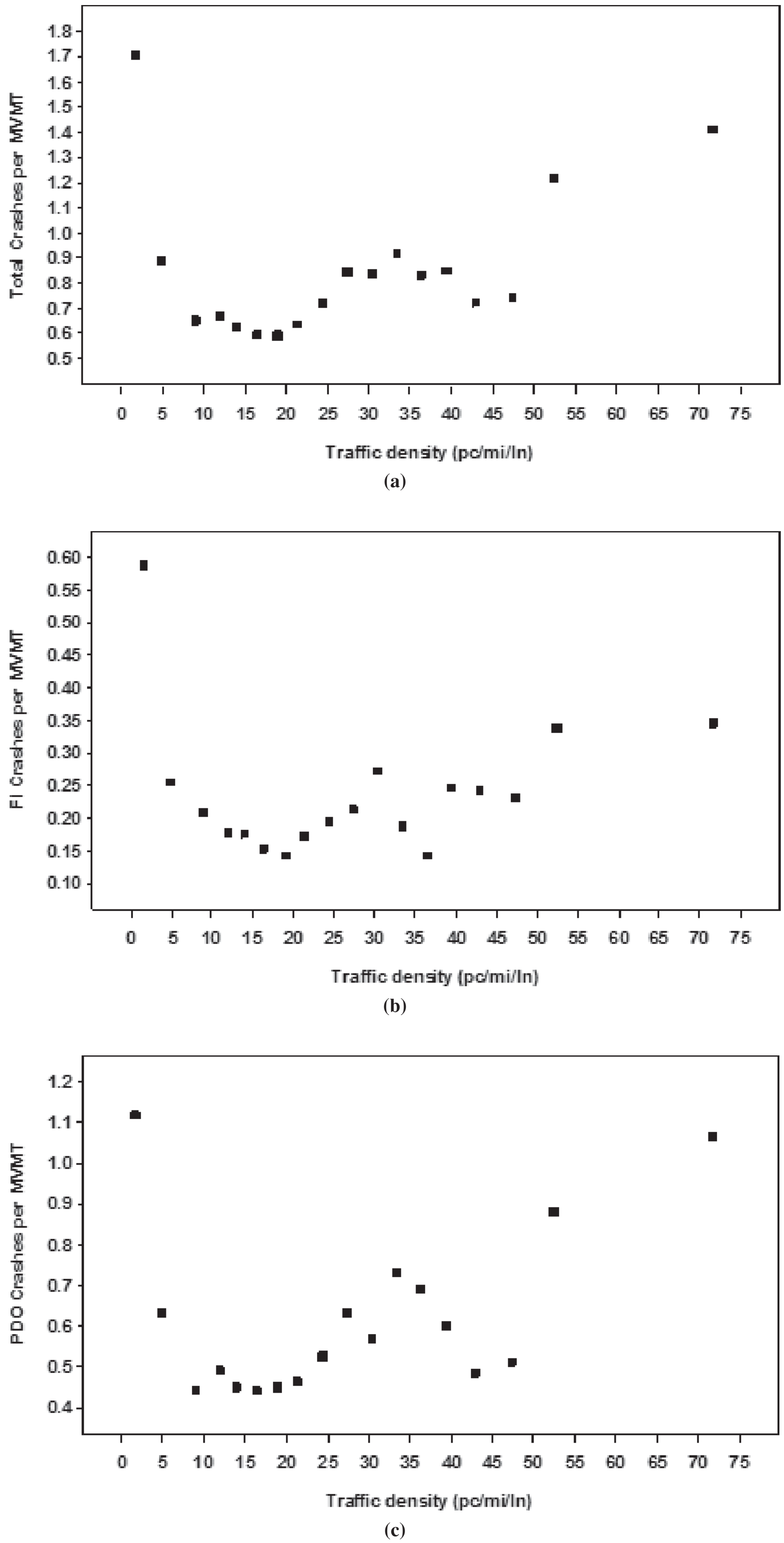


Figure 5.2. Freeway traffic density versus (a) total, (b) FI, and (c) PDO crash rates in the Minneapolis–St. Paul area.

LOS category (see Table 5.5) and the midpoint of traffic density for that LOS category. The plots generally show a U-shaped curve with the lowest crash rates in the middle of the crash rate range at about LOS C. Crash rates at lower densities (i.e., better LOS) are slightly higher than the minimum crash rate. Crash rates at higher densities (i.e., poorer LOS) are substantially higher than the minimum crash rate.

The relationships implied by Figure 5.1 appear promising to evaluate the safety effects of design treatments intended to reduce nonrecurrent congestion. For example, if a particular treatment shortens the duration of several incidents and results in 5 h per year with traffic operations in LOS C rather than LOS F, the relationships implied by Figure 5.1 should help to quantify that safety benefit as a specific number of crashes reduced.

Figure 5.2 shows a plot of crash rate and traffic density data for the Minneapolis–St. Paul area analogous to that shown for the Seattle area in Figure 5.1. The Minneapolis–St. Paul data show a relationship similar to Seattle, but the U-shaped curve is not as pronounced and is complicated by highly variable data in the traffic density range from 30 to 40 pc/mi/lane (i.e., LOS D through E+). However, regression modeling confirmed the U-shaped nature of the crash rate–traffic density relationship. There is no obvious explanation for this secondary peak, which is not present in the Seattle data and may be a quirk of the data for Minneapolis–St. Paul.

The U-shaped relationship between crash rate and traffic density has a clear interpretation. At low traffic densities, there are few vehicle–vehicle interactions, and inattentive or fatigued drivers are likely to depart from their lane or leave the roadway. This trend ameliorates as traffic densities increase to the middle range. At high traffic densities, vehicle–vehicle interactions increase to the point that rear-end or sideswipe (i.e., lane-changing) crashes become more frequent. Table 5.6 confirms that single-vehicle crashes predominate at lower traffic densities, and multiple-vehicle crashes predominate at higher traffic densities.

The crash rates were generally lower in the Minneapolis–St. Paul metropolitan area than in the Seattle metropolitan area. However, for the planned application to safety–congestion relationships, the similar shape of the two crash rate–traffic density relationships is most important. To best represent this shape, the data from the Seattle and Minneapolis–St. Paul metropolitan areas were combined, separately for each severity level, giving each area equal weight. The resulting data are shown in Figure 5.3.

The figure shows separate data for total crashes, FI crashes, and PDO crashes. Curves were fit to these data by using ordinary least squares regression analysis for the LOS range for which design treatments are of greatest interest to reduce nonrecurrent congestion (i.e., from the minimum density upward). The data suggest that the three curves start at the same density (corresponding to minimum crash rate) and have similar shapes. In modeling, it was assumed that the relationships applied would be used only in the range from the minimum observed crash rate to the highest observed density. Predicting changes in crash rate with traffic density under free-flow conditions is not relevant to the assessment of design treatments for nonrecurrent congestion. Predicting changes in crash rate substantially above the observed data for the highest density is not reliable. Regression models were obtained only for total and FI crashes; a model for PDO crashes was obtained by subtraction.

The best fit to the data was found to be a third-order polynomial with respect to density, as shown by Equation 5.5:

$$\text{Crash rate} = a_0 + a_1 \times \text{Density} + a_2 \times \text{Density}^2 + a_3 \times \text{Density}^3 \tag{5.5}$$

The regression results, based on 18 data points each for total and FI crash rates, are summarized in Table 5.7. All coefficients were statistically significant at the 0.0001 level.

The total and FI curves reach a local minimum at a density around 20 pc/mi/lane; this value was selected as the density below which the data would not be modeled. At the high end of the density range, the curves were ended at a density of 78 pc/mi/lane. The two right-hand columns in Table 5.7 present the crash rates for each severity level at the ends of the fitted curve (20 and 78 pc/mi/lane). Figure 5.4 illustrates the observed and predicted crash rates as a function of traffic density. The final relationships are shown in Equations 5.6 through 5.8.

$$\text{Total crashes per MVMT} = 2.636 - 0.2143 \times D + 0.00708 \times D^2 - 4.80 \times 10^{-5} \times D^3 \tag{5.6}$$

$$\text{FI crashes per MVMT} = 1.022 - 0.0842 \times D + 0.00264 \times D^2 - 1.79 \times 10^{-5} \times D^3 \tag{5.7}$$

$$\text{PDO crashes per MVMT} = 1.614 - 0.1301 \times D + 0.00444 \times D^2 - 3.01 \times 10^{-5} \times D^3 \tag{5.8}$$

The crash rate–traffic density relationships shown in Figure 5.4 and Equations 5.6 through 5.8 are used in two ways in the analysis of the effectiveness of design treatments for nonrecurrent congestion. The primary application is to estimate the percentage reduction in crashes expected from the reduction in congestion resulting from the implementation of any of the design treatments of interest. A secondary application is to allocate crashes between hours of the day on the basis of the congestion levels present. The

Table 5.6. Crash Type Distribution for Seattle and Minneapolis–St. Paul Freeways by LOS Category

		Level of Service[a]					
		A	B	C	D	E	F
Crash Type	**Collision Type**	**No. of Crashes (% of total)**					
Seattle							
Single vehicle	Run-off-road	56 (4.3)	26 (2.4)	26 (1.5)	17 (1.0)	6 (0.4)	7 (0.2)
	Fixed object	502 (38.4)	249 (22.9)	233 (13.9)	157 (9.1)	66 (4.4)	66 (1.8)
	Animal	17 (1.3)	4 (0.4)	6 (0.4)	3 (0.2)	0 (0.0)	0 (0.0)
	Overturn	50 (3.8)	31 (2.8)	36 (2.1)	20 (1.2)	6 (0.4)	14 (0.4)
	Pedestrian	5 (0.4)	0 (0.0)	3 (0.2)	1 (0.1)	0 (0.0)	1 (0)
	Other	62 (4.7)	34 (3.1)	34 (2.0)	27 (1.6)	7 (0.5)	15 (0.4)
	Subtotal	692 (52.9)	344 (31.6)	338 (20.1)	225 (13.1)	85 (5.6)	103 (2.8)
Multiple vehicle	Rear end	355 (27.2)	456 (41.9)	915 (54.5)	1102 (63.9)	1179 (78.2)	3115 (84.4)
	Same-direction sideswipe	95 (7.3)	96 (8.8)	179 (10.7)	192 (11.1)	135 (9.0)	276 (7.5)
	Opposite-direction sideswipe	88 (6.7)	117 (10.7)	154 (9.2)	134 (7.8)	76 (5.0)	141 (3.8)
	Head-on	3 (0.2)	3 (0.3)	0 (0.0)	2 (0.1)	1 (0.1)	1 (0.0)
	Angle	62 (4.7)	53 (4.9)	63 (3.8)	45 (2.6)	20 (1.3)	37 (1.0)
	Other	12 (0.9)	20 (1.8)	29 (1.7)	24 (1.4)	11 (0.7)	16 (0.4)
	Subtotal	615 (47.1)	745 (68.4)	1,340 (79.9)	1,499 (86.9)	1,422 (94.4)	3,586 (97.2)
Total		1,307 (100.0)	1,089 (100.0)	1,678 (100.0)	1,724 (100.0)	1,507 (100.0)	3,689 (100.0)
Minneapolis–St. Paul							
Single vehicle	Run-off-road	304 (24.4)	136 (10.9)	81 (7.5)	35 (5.6)	11 (6.0)	14 (4.9)
	Fixed object	33 (2.6)	16 (1.3)	5 (0.5)	4 (0.6)	1 (0.5)	3 (1.0)
	Animal	9 (0.7)	6 (0.5)	3 (0.3)	1 (0.2)	0 (0.0)	2 (0.7)
	Railroad train	2 (0.2)	0 (0.0)	2 (0.2)	0 (0.0)	0 (0.0)	1 (0.3)
	Parked motor vehicle	2 (0.2)	0 (0.0)	0 (0.0)	0 (0.0)	0 (0.0)	0 (0.0)
	Overturn	1 (0.1)	0 (0.0)	1 (0.1)	0 (0.0)	0 (0.0)	0 (0.0)
	Pedestrian	1 (0.1)	0 (0.0)	0 (0.0)	0 (0.0)	0 (0.0)	0 (0.0)
	Other	175 (14.0)	101 (8.1)	58 (5.4)	24 (3.8)	4 (2.2)	14 (4.9)
	Subtotal	527 (42.3)	259 (20.8)	150 (13.9)	64 (10.3)	16 (8.8)	34 (11.8)
Multiple vehicle	Rear end	327 (26.2)	576 (46.3)	640 (59.3)	423 (67.8)	114 (62.6)	193 (67.2)
	Same-direction sideswipe	191 (15.3)	207 (16.6)	142 (13.2)	60 (9.6)	22 (12.1)	25 (8.7)
	Opposite-direction sideswipe	6 (0.5)	2 (0.2)	4 (0.4)	2 (0.3)	1 (0.5)	0 (0.0)
	Head-on	17 (1.4)	13 (1.0)	8 (0.7)	1 (0.2)	0 (0.0)	0 (0.0)
	Angle	39 (3.1)	36 (2.9)	22 (2.0)	9 (1.4)	6 (3.3)	2 (0.7)
	Other	139 (11.2)	151 (12.1)	113 (10.5)	65 (10.4)	23 (12.6)	33 (11.5)
	Subtotal	719 (57.7)	985 (79.2)	929 (86.1)	560 (89.7)	166 (91.2)	253 (88.2)
Total		1,246 (100.0)	1,244 (100.0)	1,079 (100.0)	624 (100.0)	182 (100.0)	287 (100.0)

[a] LOS was assigned to each crash on the basis of the freeway segment and the traffic conditions for the 15-min period in which the crash occurred.

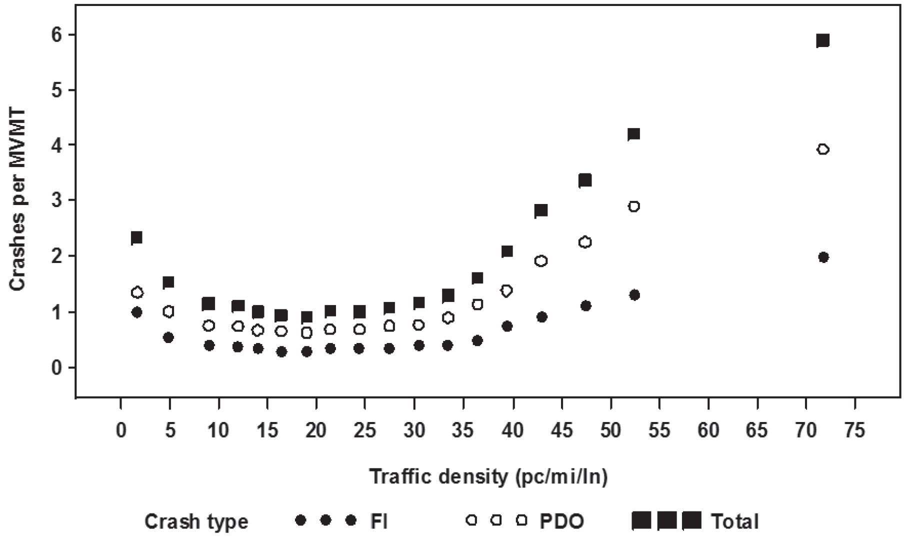


Figure 5.3. Crash rate versus density for combined Seattle and Minneapolis–St. Paul areas.

Table 5.7. Regression Results for FI and Total Crash Rates versus Density

Severity Level	Regression Coefficient				Model Fit		Crash Rate (crashes/MVMT) at Specified Density	
	a_0	a_1	a_2	a_3	RMSE	R^2 (%)	20 pc/mi/lane	78 pc/mi/lane
Total	2.636	–0.2143	0.00708	-4.80×10^{-5}	0.183	98.5	0.80	6.22
FI	1.022	–0.0842	0.00264	-1.79×10^{-5}	0.072	98.0	0.25	2.04
PDO	1.614	–0.1301	0.00444	-3.01×10^{-5}	NA	NA	0.54	4.17

Note: For PDO crashes, regression coefficients and crash rates for 20 and 78 pc/mi/lane were obtained by subtraction (total – FI).
RMSE = root mean square error.

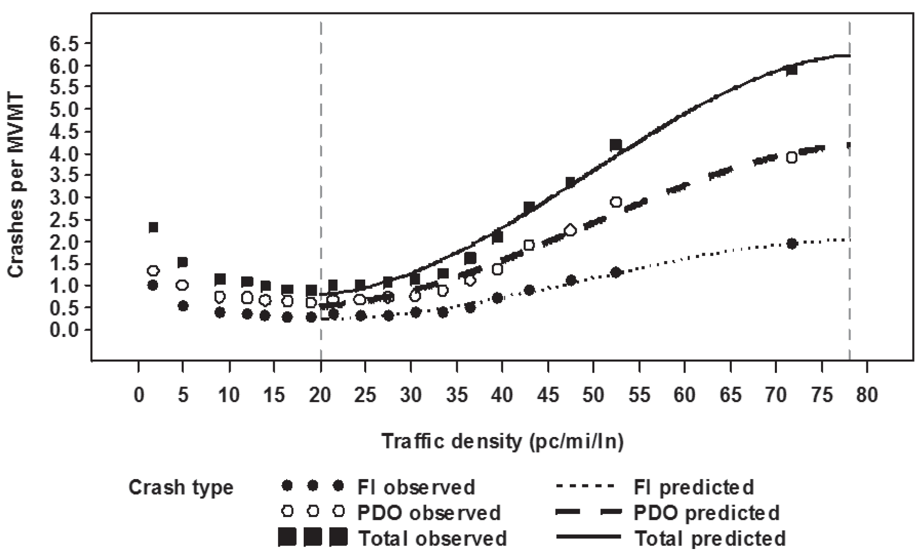


Figure 5.4. Observed and predicted total, FI, and PDO crash rates versus traffic density.

relationships shown in Figure 5.4 and Equations 5.6 through 5.8 are applied only in the traffic density range from 20 to 78 pc/mi/lane. Above and below this crash density range, the crash rate is assumed to be constant at the end-point values shown in the two right-hand columns of Table 5.7. The full crash rate–traffic density relationship incorporated in the assessment methodology is shown by Equations 5.9, 5.10, and 5.11:

$$\text{Total crashes per MVMT} = \begin{cases} 0.80 & \text{if Density} < 20 \text{ pc/mi/lane} \\ 2.636 - 0.2143 \times D + 0.00708 & \\ \times D^2 - 4.80 \times 10^{-5} \times D^3 & \\ 6.22 & \text{if Density} > 78 \text{ pc/mi/lane} \end{cases} \tag{5.9}$$

$$\text{FI crashes per MVMT} = \begin{cases} 0.25 & \text{if Density} < 20 \text{ pc/mi/lane} \\ 1.022 - 0.0842 \times D + 0.00264 & \\ \times D^2 - 1.79 \times 10^{-5} \times D^3 & \\ 2.04 & \text{if Density} > 78 \text{ pc/mi/lane} \end{cases} \tag{5.10}$$

$$\text{PDO crashes per MVMT} = \begin{cases} 0.54 & \text{if Density} < 20 \text{ pc/mi/lane} \\ 1.614 - 0.1301 \times D + 0.00444 & \\ \times D^2 - 3.01 \times 10^{-5} \times D^3 & \\ 4.17 & \text{if Density} > 78 \text{ pc/mi/lane} \end{cases} \tag{5.11}$$

The two applications in which the crash rate–traffic density relationships are used in the assessment tool are described in more detail below.

Prediction of Crash Reduction Due to Congestion Reduction Resulting from Design Treatments

For each design treatment evaluated in Project L07, an untreated travel time index (TTI) curve and a treated TTI curve were predicted for each hour of the day. The Project L07 research team devised a methodology to convert a TTI curve into an equivalent traffic density distribution that could use the safety–density relationship to predict untreated and treated crash rates. This methodology is described in the following paragraphs.

Project L03 provided equations to predict the 10th, 50th, 80th, 95th, and 99th percentile values of the cumulative TTI distribution. The lowest or zero percentile value of TTI is also

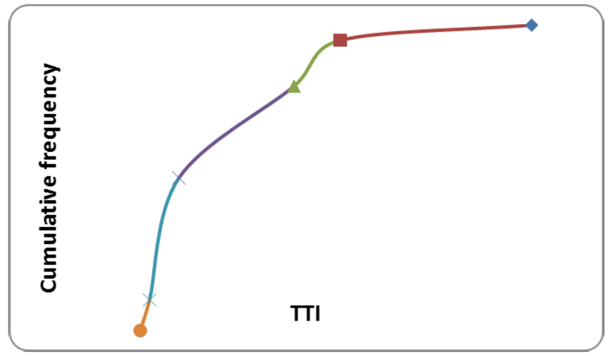


Figure 5.5. Example of cumulative TTI distribution approximated from five percentile values.

known because it is, by definition, equal to 1.00. As shown in Figure 5.5, these five percentile values can be plotted to estimate the cumulative TTI curve.

To estimate the average density from this TTI curve, the data are divided into five subsets. Each subset represents a proportion of all the vehicles using the freeway section during the specific hour under consideration (e.g., 8:00 to 9:00 a.m.). These proportions are termed the "weight" of each subset, and are as follows:

- Subset 1 ($TTI_0 - TTI_{10}$): $Weight_1 = 10\%$
- Subset 2 ($TTI_{10} - TTI_{50}$): $Weight_2 = 40\%$
- Subset 3 ($TTI_{50} - TTI_{80}$): $Weight_3 = 30\%$
- Subset 4 ($TTI_{80} - TTI_{95}$): $Weight_4 = 15\%$
- Subset 5 ($TTI_{95} - TTI_{99}$): $Weight_5 = 5\%$

Subset 5 is given a weight of 5%, even though the difference between 95% and 99% is only 4%. This assumption is equivalent to estimating that $TTI_{100} \approx TTI_{99}$.

Each subset has an average TTI value representing the travel time for all the vehicles in the subset. This average TTI value is calculated for each subset by using Equation 5.12:

$$TTI_{\text{subset } i} = \frac{(TTI_{\text{lower}} + TTI_{\text{upper}})}{2} \tag{5.12}$$

where

$TTI_{\text{subset } i}$ = average TTI value for subset i;
TTI_{lower} = lowest TTI value for the subset; and
TTI_{upper} = highest TTI value for the subset.

For example, for subset 1 ($TTI_0 - TTI_{10}$), $TTI_{\text{lower}} = TTI_0$, and $TTI_{\text{upper}} = TTI_{10}$.

By using these values, the average travel time can be calculated for each subset. This value represents the amount of time that one vehicle would spend on the freeway section if that vehicle had a TTI equal to the average TTI for the subset, as shown by Equation 5.13:

$$TT_{\text{subset } i} = \frac{\text{Length} \times TTI_{\text{subset } i}}{FFS} \quad (5.13)$$

where

$TT_{\text{subset } i}$ = average travel time for subset i (h);
Length = length of freeway segment (mi);
$TTI_{\text{subset } i}$ = average TTI value for the subset; and
FFS = free-flow speed for the freeway segment (mph).

The free-flow speed is determined by using HCM, Chapter 23 (*3*) procedures (see Equation 5.26). The average speed for each subset is then calculated as shown by Equation 5.14:

$$\text{Speed}_{\text{subset } i} = \frac{\text{Length}}{TT_{\text{subset } i}} = \frac{FFS}{TTI_{\text{subset } i}} \quad (5.14)$$

where

$\text{Speed}_{\text{subset } i}$ = average speed for subset i (mph);
Length = length of freeway segment (mi); and
$TT_{\text{subset } i}$ = average travel time for the subset (h).

Next, the density is calculated for each subset as shown by Equation 5.15. Because the safety–density relationship is only valid for densities between 20 and 78 pc/mi/lane, calculated densities below or above this range are limited at 20 and 78, respectively.

$$\text{Density}_{\text{subset } i} = (225)\left(1 - \frac{\text{Speed}_{\text{subset } i}}{FFS}\right) = (225)\left(1 - \frac{1}{TTI_{\text{subset } i}}\right) \quad (5.15)$$

where $\text{Density}_{\text{subset } i}$ is the average traffic density (pc/mi/lane) for subset *i*, and $\text{Speed}_{\text{subset } i}$ is the average speed (mph) for the subset.

The FI crash rate and PDO crash rate for each subset are estimated by using the safety–density relationship, as shown by Equations 5.16 and 5.17, respectively:

$$\begin{aligned} FICR_{\text{subset } i} = 1.022 &- 0.0842(\text{Density}_{\text{subset } i}) \\ &+ 0.00264(\text{Density}_{\text{subset } i})^2 \\ &- 0.0000179(\text{Density}_{\text{subset } i})^3 \end{aligned} \quad (5.16)$$

where $FICR_{\text{subset } i}$ is the FI crash rate (crashes/MVMT) for subset *i*.

$$\begin{aligned} PDOCR_{\text{subset } i} = 1.614 &- 0.1301(\text{Density}_{\text{subset } i}) \\ &+ 0.00444(\text{Density}_{\text{subset } i})^2 \\ &- 0.0000179(\text{Density}_{\text{subset } i})^3 \end{aligned} \quad (5.17)$$

where $PDOCR_{\text{subset } i}$ is the PDO crash rate (crashes/MVMT) for subset *i*.

To estimate the crash frequencies from the crash rates, the annual travel must be determined, which is calculated as shown by Equation 5.18:

$$AMVMT_{tot} = \frac{\text{Demand} \times \text{Length} \times N_{days}}{1{,}000{,}000} \quad (5.18)$$

where

$AMVMT_{tot}$ = total annual million vehicle miles traveled (MVMT/year);
Demand = hourly volume for the freeway segment during an hour time-slice (vehicles/h);
Length = length of freeway segment (mi); and
N_{days} = number of days in yearly study period.

As explained in Chapter 4 under Model Variables, the usual days considered in the yearly study period are nonholiday weekdays, so that N_{days} is 250 days.

The annual travel for each subset is then calculated as given by Equation 5.19:

$$AMVMT_{\text{subset } i} = (AMVMT_{tot})(\text{Weight}_{\text{subset } i}) \quad (5.19)$$

where $AMVMT_{\text{subset } i}$ equals the annual million vehicle miles traveled for subset *i* (MVMT/year), and $\text{Weight}_{\text{subset } i}$ is the proportion of all vehicles using the freeway section during the hour.

The total predicted number of crashes can then be calculated for the freeway segment by summing the predicted number of crashes in each subset. The number of predicted FI and PDO crashes are calculated as given by Equations 5.20 and 5.21, respectively:

$$NFI_{tot} = \sum_{i=1}^{5} (FICR_{\text{subset } i})(AMVMT_{\text{subset } i}) \quad (5.20)$$

where NFI_{tot} is the total predicted number of FI crashes per year.

$$NPDO_{tot} = \sum_{i=1}^{5} (PDOCR_{\text{subset } i})(AMVMT_{\text{subset } i}) \quad (5.21)$$

where $NPDO_{tot}$ is the total predicted number of PDO crashes per year.

The final values for the predicted number of FI and PDO crashes per year are calculated and recorded first for the untreated TTI curve on the basis of the five TTI percentiles. Next, this series of calculations is completed using the five percentile values for the treated TTI curve. With these values, the reductions in FI and PDO crashes can be estimated as shown by Equations 5.22 and 5.23, respectively:

$$\%\text{Reduction}_{\text{FI}} = \left(1 - \frac{\text{NFI}_{\text{tot tr}}}{\text{NFI}_{\text{tot unt}}}\right) * 100 \qquad (5.22)$$

where

$\%\text{Reduction}_{\text{FI}}$ = estimated percentage reduction in fatal-and-injury crashes due to the congestion-mitigation effect of a design treatment;
$\text{NFI}_{\text{tot unt}}$ = untreated total predicted number of fatal and major-injury crashes per year; and
$\text{NFI}_{\text{tot tr}}$ = treated total predicted number of fatal and major-injury crashes per year.

$$\%\text{Reduction}_{\text{PDO}} = \left(1 - \frac{\text{NPDO}_{\text{tot tr}}}{\text{NPDO}_{\text{tot unt}}}\right) * 100 \qquad (5.23)$$

where

$\%\text{Reduction}_{\text{PDO}}$ = estimated percentage reduction in PDO crashes due to the congestion-mitigation effect of a design treatment;
$\text{NPDO}_{\text{tot unt}}$ = untreated total predicted number of PDO crashes per year; and
$\text{NPDO}_{\text{tot tr}}$ = treated total predicted number of PDO crashes per year.

Finally, the percentage reduction values for each crash type are multiplied by the number of expected crashes for the roadway segment to determine the expected number of crashes reduced, as shown for FI and PDO crashes, respectively, in Equations 5.24 and 5.25:

$$\text{NReduction}_{\text{FI}} = \left(\frac{\%\text{Reduction}_{\text{FI}}}{100}\right)(\text{Nexp}_{\text{FI}}) \qquad (5.24)$$

where $\text{NReduction}_{\text{FI}}$ is the predicted number of fatal-and-injury crashes per year to be reduced by the congestion-mitigation effect of a design treatment, and Nexp_{FI} is the number of expected fatal and major-injury crashes per year without treatment.

$$\text{NReduction}_{\text{PDO}} = \left(\frac{\%\text{Reduction}_{\text{PDO}}}{100}\right)(\text{Nexp}_{\text{PDO}}) \qquad (5.25)$$

where $\text{NReduction}_{\text{PDO}}$ is the predicted number of PDO crashes per year to be reduced by the congestion-mitigation effect of a design treatment, and Nexp_{PDO} is the number of expected PDO crashes per year without treatment.

The estimated number of FI and PDO crashes reduced per year by a particular design treatment at a particular site can then be used in a life-cycle benefit–cost analysis to quantify the value of the annual safety benefit expected from the design treatment. The life-cycle benefit–cost analysis methodology is presented in Chapter 6 of this report.

Estimation of Crash Distributions by Hour of Day

Chapter 23 of the 2000 *Highway Capacity Manual* provides a methodology for estimating freeway operating speed (*3*). To determine the operating speed, the free-flow speed of the freeway segment is first calculated using Equation 5.26, which is based on HCM Equation 23-1:

$$\text{FFS} = \text{BFFS} - f_{\text{LW}} - f_{\text{LC}} - f_N - f_{\text{ID}} \qquad (5.26)$$

where

FFS = free-flow speed (mph);
BFFS = base FFS (70 mph, urban; 75 mph, rural);
f_{LW} = adjustment for lane width from HCM Exhibit 23-4 (mph);
f_{LC} = adjustment for right-shoulder lateral clearance from HCM Exhibit 23-5 (mph);
f_N = adjustment for number of lanes from HCM Exhibit 23-6 (mph); and
f_{ID} = adjustment for interchange density from HCM Exhibit 23-7 (mph).

Operating speed for $70 < \text{FFS} \leq 75$ and a flow rate of $(3{,}400 - 30\text{FFS}) < v_p \leq 2{,}400$ is given by Equation 5.27:

$$S = \text{FFS} - \left[\left(\text{FFS} - \frac{160}{3}\right)\left(\frac{v_p + 30\text{FFS} - 3{,}400}{30\text{FFS} - 1{,}000}\right)^{2.6}\right] \qquad (5.27)$$

where S is operating speed (mph), and v_p is the 15-min passenger car–equivalent flow rate (passenger cars per hour per lane [pcphpl]).

For $55 < \text{FFS} \leq 70$ and $(3{,}400 - 30\text{FFS}) < v_p \leq (1{,}700 + 10\text{FFS})$, S is given by Equation 5.28:

$$S = \text{FFS} - \left[\frac{1}{9}(7\text{FFS} - 340)\left(\frac{v_p + 30\text{FFS} - 3{,}400}{40\text{FFS} - 1{,}700}\right)^{2.6}\right] \qquad (5.28)$$

For $55 \leq \text{FFS} \leq 75$ and $v_p \leq (3{,}400 - 30\text{FFS})$, operating speed equals free-flow speed, as shown by Equation 5.29:

$$S = \text{FFS} \qquad (5.29)$$

The average density can then be estimated by dividing operating speed S by the hourly demand volume, as shown by Equation 5.30:

$$\text{Density}_{\text{hour } i} = \frac{S}{\text{Demand}_{\text{hour } i}} \qquad (5.30)$$

where $\text{Density}_{\text{hour } i}$ is the average traffic density for hour i (pc/mi/lane), and $\text{Demand}_{\text{hour } i}$ is the hourly demand volume for hour i (pc/h).

Using this traffic density estimate, the crash rate–traffic density relationship developed in Project L07 predicts the crash rate for each hourly time-slice by using Equation 5.31:

$$CR_{\text{hour } i} = 2.636 - 0.2143\left(\text{Density}_{\text{hour } i}\right) + 0.00708\left(\text{Density}_{\text{hour } i}\right)^2 - 0.000048\left(\text{Density}_{\text{hour } i}\right)^3 \quad (5.31)$$

where $CR_{\text{hour } i}$ is the total crash rate for hour i (crashes/MVMT).

The total predicted number of crashes for each hourly time-slice is then estimated as shown by Equation 5.32:

$$NC_{\text{hour } i} = \frac{\left(\text{Demand}_{\text{hour } i}\right)(\text{Length})(250)\left(CR_{\text{hour } i}\right)}{1{,}000{,}000} \quad (5.32)$$

where $NC_{\text{hour } i}$ is the predicted total number of crashes for hour i (crashes/year), and Length is the length of the freeway segment (mi).

Finally, the estimated number of crashes for each hour is summed across all 24 hourly time-slices to determine the total number of predicted crashes for the year. Each hour's predicted number of crashes is then divided by the total number of predicted crashes to determine the relative probability of a crash occurring during that hour, as described by Equation 5.33:

$$\text{CrashProb}_{\text{hour } i} = \frac{NC_{\text{hour } i}}{\sum_{i=1}^{24} NC_{\text{hour } i}} \quad (5.33)$$

where $\text{CrashProb}_{\text{hour } i}$ is the relative probability of a crash during hour i.

CHAPTER 6

Life-Cycle Benefit–Cost Analysis

A methodology was developed for conducting a life-cycle benefit–cost evaluation for the design treatments considered in this research. The method uses expected improvements in travel time, travel time reliability, and safety to estimate monetary benefits of treatment installation and compares those benefits to the expected costs of implementation and maintenance of the design treatment. This section describes the methodology for determining the values of these benefits and costs and describes the calculation procedure to estimate the final benefit–cost ratio. A spreadsheet tool to implement this methodology is described in Chapter 2 under Analysis Tool.

Overview of Life-Cycle Benefit–Cost Analysis Methodology

The life-cycle benefit–cost analysis methodology is intended to obtain two measures that compare the benefits and costs of design treatments expressed in monetary terms: the benefit–cost ratio and net present benefits. These two measures are defined by Equations 6.1 and 6.2:

$$\text{Benefit–Cost Ratio} = B/C \qquad (6.1)$$

$$\text{Net Present Benefits} = B - C \qquad (6.2)$$

where B is the present value of treatment benefits ($), and C is the present value of treatment costs ($).

These measures can be used to assess whether a specific design treatment has positive net benefits for application at a given site (i.e., if $B/C > 1$ or $B - C > 0$), and they can also be used to compare the cost-effectiveness of alternative treatments. Any specific treatment is evaluated over its service life (i.e., the period of time over which the treatment will continue to provide benefits without renewal, reconstruction, or replacement). When alternative treatments with differing service lives are compared, that comparison needs to be conducted over multiple renewal cycles for one or both treatments. The analysis period is typically the least common multiple of the service lives of the design treatments being compared. For example, comparison of a design treatment with a 10-year service life to a treatment with a 15-year service life would need to be conducted with a 30-year analysis period (i.e., three life cycles for the first treatment and two life cycles for the second treatment). This comparison of treatments over multiple life cycles is why this type of analysis is referred to as life-cycle benefit–cost analysis.

The costs of design treatments are determined by combining the initial implementation or construction cost and the annual maintenance cost, as shown in Equation 6.3:

$$C = \text{IC} + \text{AMC}\,(\text{USPWF}) \qquad (6.3)$$

where

C = design treatments costs ($);
IC = implementation or construction cost ($);
AMC = annual maintenance cost ($); and
USPWF = uniform series present worth factor.

The uniform series present worth factor is defined by Equation 6.4:

$$\text{USPWF} = \frac{(1+i)^n - 1}{i(1+i)^n} \qquad (6.4)$$

where i is the minimum attractive rate of return or discount rate, expressed as a proportion (e.g., $i = 0.04$ represents a 4% discount rate); and n is the service life of design treatment in years.

The benefits of design treatments in the life-cycle benefit–cost analysis combine both traffic operational and safety benefits, as shown by Equation 6.5:

$$B = (\text{AOB} + \text{ASB})\,\text{USPWF} \qquad (6.5)$$

where AOB is the annual traffic operational benefit ($), and ASB is the annual safety benefit ($).

Equation 6.5 is suitable for the current assessment tool, which is based on constant traffic volumes. A potential enhancement of the tool would allow the user to specify an annual percentage growth in traffic volume. Equation 6.5 would then be replaced by Equation 6.6:

$$B = \sum_{j=1}^{n} \left(\text{AOB}_j + \text{ASB}_j\right)\left(\frac{1}{(1+i)^j}\right) \tag{6.6}$$

where

AOB_j = annual traffic operational benefit for year j (\$);
ASBj = annual safety benefit for year j (\$); and
$1/(1+i)^j$ = single-amount present worth factor for year j.

The annual traffic operational benefit for a design treatment is determined as shown by Equation 6.7:

$$\text{AOB} = \sum_{k=1}^{24} \Delta D_k\,(\text{VOT}) + \Delta\sigma_k\,(\text{VOR})\, V_k N_d L \tag{6.7}$$

where

ΔD_k = change in annual traffic operational delay due to the design treatment during hour k (vehicle hour);
VOT = value of travel time (\$/vehicle hour);
$\Delta\sigma_k$ = change in the standard deviation of travel time during hour k;
VOR = value of reliability (\$/vehicle hour);
V_k = traffic volume on facility during hour k;
N_d = number of days per year (= 250 days); and
L = roadway segment length (mi).

This approach to assessing the value of travel time and reliability is based directly on the current state of knowledge about the value of reliability. It may be appropriate to update this approach as the state of knowledge evolves. In addition, a possible enhancement of the tool could incorporate additional operational benefits, such as vehicle operating cost, including and fuel cost, savings and reduced emissions.

The annual safety benefit for a design treatment is determined as shown by Equation 6.8:

$$\begin{aligned}\text{ASB} = {} & \text{NReduction}_{\text{FI}}\,(\text{CC}_{\text{FI}}) + \text{NReduction}_{\text{PDO}}\,(\text{CC}_{\text{PDO}}) \\ & + \text{DSB}_{\text{FSI}}\,(\text{CC}_{\text{FSI}}) + \text{DSB}_{\text{MI}}\,(\text{CC}_{\text{MI}}) \\ & + \text{DSB}_{\text{PDO}}\,(\text{CC}_{\text{PDO}})\end{aligned} \tag{6.8}$$

where

$\text{NReduction}_{\text{FI}}$ = predicted number of fatal-and-injury crashes per year to be reduced by the congestion-mitigation effect of a design treatment;
$\text{NReduction}_{\text{PDO}}$ = predicted number of property-damage-only crashes per year to be reduced by the congestion-mitigation effect of a design treatment;
CC_{FI} = crash cost savings per fatal-and-injury crash reduced (\$);
CC_{FSI} = crash cost savings per fatal-and-severe-injury crash reduced (\$);
CC_{MI} = crash cost savings per minor-injury crash reduced (\$);
CC_{PDO} = crash cost savings per property-damage-only crash reduced (\$);
DSB_{FSI} = annual number of fatal-and-severe-injury crashes reduced as a direct safety benefit of the design treatment;
DSB_{MI} = annual number of minor-injury crashes reduced as a direct safety benefit of the design treatment; and
DSB_{PDO} = annual number of property-damage-only crashes reduced as a direct benefit of the design treatment.

The crash severity levels used in the benefit–cost analysis are derived from the KABCO scale of crash severity levels (K = killed; A = incapacitating injury; B = nonincapacitating injury; C = possible injury; O = no injury; and U = injured, severity unknown) for which the Federal Highway Administration has developed crash cost estimates (*9*). Severe-injury crashes, as this term is used in the benefit–cost analysis, are equivalent to incapacitating injury crashes (also known as A-injury crashes in the KABCO scale). Fatal and severe-injury crashes are combined in the benefit–cost analysis because, if fatal crashes were considered alone, the random occurrence of a single fatal crash might influence the analysis results too strongly. Minor-injury crashes include both nonincapacitating injury crashes (also known as B-injury crashes) and possible-injury crashes (also known as C-injury crashes).

The safety benefits from the congestion reduction effects of the safety treatments are represented in Equation 6.8 by the terms $\text{NReduction}_{\text{FI}}$ and $\text{NReduction}_{\text{PDO}}$. The methodology for deriving these terms has been presented in Chapter 5 in Equations 5.12 through 5.25.

Each of the individual terms of the life-cycle benefit–cost methodology is discussed below.

Implementation or Construction Cost

The implementation or construction cost for a design treatment is the initial one-time cost to install or construct that treatment. This input to the assessment methodology is provided by the user. Highway agencies generally have good information on the cost of implementing treatments.

Annual Maintenance Cost

The annual maintenance cost for a design treatment is the recurring yearly cost of maintaining the design treatment in place. Depending on the nature of the treatment, these costs could be incurred by either highway agency maintenance forces or contractors and could be either recurring costs to keep the treatment in repair or per incident costs to deploy the treatment or restore it after use. Annual maintenance costs are supplied by the user as an input for the assessment methodology.

Minimum Attractive Rate of Return or Discount Rate

The minimum attractive rate of return or discount rate (i in Equation 6.4) represents the time value of capital invested in design treatments to reduce nonrecurrent congestion and improve reliability. The discount rate is used to reduce future costs and benefits to their present values so they can be compared on a common basis. The suggested default value of the discount rate is 7%. This value of the discount rate was chosen on the basis of Office of Management and Budget Circular A-94 (*10*), which specifies a real discount rate of 7% for analysis of public investments. Circular A-94, which has been U.S. government policy since 1992, has been reissued within the last year with the discount rate provision unchanged.

Service Life

The service life (n in Equation 6.4) of design treatments varies over a broad range from five (or fewer) to 20 years (or more). It is possible that, due to traffic volume growth, some design treatments may lose their effectiveness in reducing nonrecurrent congestion before the end of their physical life. Such treatments may be considered to become functionally obsolescent. This possibility should be considered in choosing the service life for a treatment.

Change in Annual Traffic Operational Delay

The change in annual traffic operational delay for a specific design treatment during a specific hourly time-slice (ΔD_k in Equation 6.7; also referred to as ΔLI_k in Equation 4.39) is computed with a procedure documented in Chapter 4. ΔD_k is derived directly from the area between the treated and untreated travel time index (TTI) curves using the approximation shown in Figure 4.7 and Equation 4.40. Once the treated and untreated TTI curves have been established for a design treatment, the computation of ΔD_k using the procedure based on Figure 4.7 and Equation 4.40 is performed in the same way for every design treatment. The methods for determining the treated TTI curves vary by design treatment and are illustrated in Chapter 4 under Quantifying Design Treatment Effects on Reliability by Using the Cumulative TTI Curve.

Change in the Standard Deviation of Travel Time

The standard deviation of travel time for a specific design treatment during a specific hourly time-slice ($\Delta\sigma_k$ in Equation 6.7) is computed with a procedure documented in Chapter 4 under Change in Variance. $\Delta\sigma_k$ represents the difference between the standard deviations of the treated and untreated TTI curves, like the example curves shown in Figure 4.4. The standard deviation of either the treated or untreated TTI curve can be determined with the approximation shown in Figure 4.8 and Equation 4.42. $\Delta\sigma_k$ is then determined as the difference between those standard deviations, as shown by Equation 6.9:

$$\Delta\sigma_k = \sigma_{\text{untreated},k} - \sigma_{\text{treated},k} \tag{6.9}$$

where

$\sigma_{\text{untreated},k}$ = standard deviation of travel time (h) for the untreated condition, derived from an untreated TTI curve like that shown in Figure 4.4; and

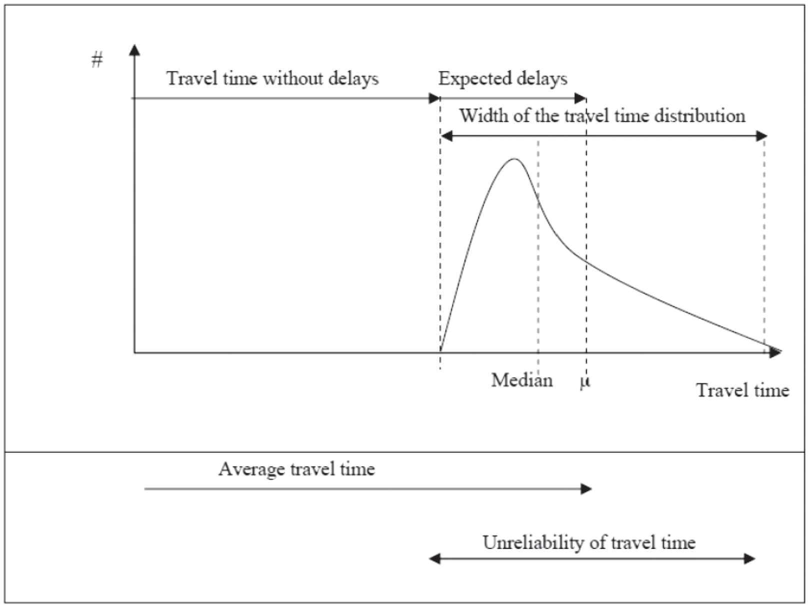
$\sigma_{\text{treated},k}$ = standard deviation of travel time (h) for the treated condition, derived from a treated TTI curve for a design treatment, like that shown in Figure 4.4.

Values of Travel Time and Reliability

This section presents the approach used in the Analysis Tool to quantify the value of reliability. Figure 6.1 illustrates a typical travel time distribution curve shown by Warffemius (*11*). The distribution is skewed with a relatively long tail toward higher travel times, which is typical of data for unreliable conditions. The mean travel time shown in Figure 6.1 represents the travel time for the average motorist. The difference between the mean travel time and the ideal or free-flow travel time (labeled in the figure as the travel time without delays) represents the average delay to motorists under the prevailing conditions.

Value of Travel Time and Delay

In economic studies, the value that a person places on his or her time spent traveling can be determined based on revealed preference or stated preference studies. In a revealed preference study, the subjects indicate the trade-offs they are willing to make between time and money through real-life decisions. These types of studies can be very difficult to set up and measure. In a stated preference survey, respondents are presented choices that help researchers determine their willingness to trade money for time and vice versa. These studies are much



Source: Warffemius (*11*).

Figure 6.1. Typical example of travel time distribution curve used to estimate delay and reliability.

simpler to conduct, and it is assumed that respondents' stated preferences would be close to their revealed preferences in most cases.

In transportation benefit–cost studies, the value of travel time and, thus, the value of delay reduction is typically considered to be a percentage of the prevailing wage, with different percentages assigned to the various trip types. The primary division of trip type is between work trips and nonwork trips. Work trips are those that are conducted on the job, in which the cost to the employer is the total of the wage and benefits of the driver, plus the same for any employee passengers. Freight trips, as a subcategory of work trips, may have additional costs per hour if the freight is time sensitive, as in the case of perishable goods.

Generally, nonwork trips are valued at a lower rate than work trips. For nonwork trips, the value of time may be greater for the driver than for the passenger, since the passenger could participate in other activities while in the car and is not required to dedicate their time to the task of driving. Passengers who are children also have a lower value of time, since their time cannot be converted into wages. Nonwork trips may also be categorized by trip purpose, such as commuting to work, commuting from personal errands, and leisure trips, as these may all have different values.

Because the value of time will vary from person to person and from trip to trip, simplifying assumptions need to be made to determine travel time savings benefits for a specific treatment on a given roadway, as users and trip types will be diverse over the life cycle of the treatment. Table 6.1 summarizes the results of studies on the value of travel time and delay reduction. Based on a review of these studies, Concas and Kolpakov (*12*) made the following recommendations concerning the value of travel time and delay reduction for use in benefit–cost studies:

- Personal travel time (including commuter travel) should be valued at 50% of the prevailing wage rate.
- On-the-clock paid travel (e.g., commercial vehicle driver) should be valued at 100% of the driver's wages plus benefits.
- The use of the national average wage rate is recommended as the basis for determining the value of time unless reliable information on the earnings of particular users of a transportation facility is available and these earnings are significantly different from the national average.

The most recent available estimate (May 2009) of the national average wage rate from the Bureau of Labor Statistics is $20.90/h (*27*).

The default value used for the value of travel time in the Analysis Tool is $15.68/h. Users may replace this value with any value considered more appropriate for their local condition.

Value of Reliability

Warffemius (*11*) makes the case that variability (i.e., the variance or standard deviation of the travel time distribution) is a useful measure of reliability. The greater the variance or

Table 6.1. Results of Studies on the Value of Travel Time and Delay Reduction (11)

Study	Year	Data Used	VOT Estimate
Becker (*13*)	1965		40% of wage rate
Beesley (*14*)	1965	Data from survey of government employees in London, United Kingdom	31% to 50% of wage rate
Lisco (*15*)	1967		20% to 51% of wage rate
Miller (*16*)	1989	Survey of multiple route choice models	60% of gross wage (on average)
Small (*17*)	1992	Values derived from multiple mode choice transportation models	20% to 100% of gross wage; 50%-reasonable average
Waters (*18*)	1992	Travel data from British Columbia, Canada	50% to 100% average wage rate for personal travel, depending on LOS; 120% to 170% of average wage rate for commercial travel, depending on LOS
Waters (*19*)	1996	Travel data from 15 commuting studies in North America	40% to 50% of after-tax wage rate (mean: 59% of after-tax wage rate; median: 42% of wage rate)
Calfee and Winston (*20*)	1998	Data from National Family Opinion survey covering commuters from major U.S. metropolitan areas	14% to 26% of gross wage; 19% of wage-average estimate
Small and Yan (*21*)	2001	Data on commute travelers on SR-91 in California	Average VOT is $22.87/h, or 72% of sample wage rate
Brownstone and Small (*22*)	2003	Travel data from ETC facilities in HOT lanes on SR-91 and I-15 in Southern California	VOT saved on the morning commute: $20 to $40 per hour, or 50% to 90% of average wage rate in the sample
U.S. DOT (*9*)	2003	Estimates are based on multiple sources of data	50% to 120% of the wage rate depending on type of travel (personal versus business); 50% of wage rate for personal local travel; 100% of wage rate for commercial local travel
Small et al. (*23*)	2005	Travel from SR-91 in greater Los Angeles, Calif., area, collected over 10-month period in 1999 to 2000	Median VOT is $21.46/h or 93% of average wage rate
Tseng et al. (*24*)	2005	Data collected in June 2004 for Dutch commuters who drive to work two or more times per week	Mean VOT for all travelers: 10 euros/h (approximately $12.10/h)
Litman (*25*)	2007	Results are drawn from multiple travel time studies	25% to 50% of prevailing wage (for personal travel)
Tilahun and Levinson (*26*)	2007	Data from stated preference survey of travelers on I-394 in Minneapolis–St. Paul, Minn., area	$10.62/h for MnPass (ETC system) subscribers who were early/on-time $25.42/h for MnPass subscribers who were late $13.63/h for nonsubscribers who were early/on-time $10.10/h for nonsubscribers who were late

Note: VOT = value of time; LOS = level of service; ETC = electronic toll collection; HOT = high-occupancy toll; DOT = Department of Transportation.

standard deviation of the travel time distribution, the greater the unreliability of travel times.

Warffemius indicates that the value of reliability can be expressed as a multiple of the value of travel time, with that multiplier referred to as the *reliability ratio,* as shown by Equation 6.10:

$$\text{VOR} = \rho\ \text{VOT} \tag{6.10}$$

where ρ is the reliability ratio.

Warffemius indicates that Copley et al. (*28*) estimated the reliability ratio as equal to 1.3 on the basis of a stated preference survey among commuters in Manchester, United Kingdom, who used their car as solo drivers on their journey to work. Copley et al. defined the reliability ratio explicitly as the "value of 1 minute of standard deviation"/"value of 1 minute of travel time." The method for estimating the standard deviation of travel time presented previously in this chapter under Change in the Standard Deviation of Travel Time can be used to implement this concept.

Warffemius (*11*) states that the average travel time and its variation (i.e., standard deviation) can be presented in stated preference surveys in such a way that these attributes are not correlated. As a consequence, the economic benefits of travel time savings and reliability improvements can be added together without the risk of double counting. This supports the combination of these two types of benefits by addition, as shown in Equation 6.8.

Table 6.2 presents a broader set of research results that have quantified the reliability ratio. As with travel time, travel time reliability is valued differently depending on the trip type and the person making the trip. For example, when driving to the airport to catch a flight, reliability is highly valuable, since unexpected delay can result in a missed flight. The value of reliability of a morning commute depends on the importance of arriving at a certain time: some jobs have set start times for which late arrivals can have significant consequences, but other jobs have flexible start times and a late arrival has a much smaller impact. The reliability of the evening commute has a lower value for most people because the arrival time at

Table 6.2. Results of Studies on the Value of Reliability

Study Authors	No. of Respondents	Trip Type		Reliability Ratio
Copley et al. (*28*)	167	Mostly work commutes		1.3
Black and Towriss (*31*)	354	Car travelers		0.79
Small et al. (*32, 33*)	NA	Commute to work		1.3
Halse and Killi (*34*)	505	Shippers		0.68
Parsons Brinckerhoff (*35*)	NA	High income (60K+)	To work	0.8
			From work	0.6
			Nonwork	0.4
		Low income (<60K)	To work	1.0
			From work	0.3
			Nonwork	0.2
	NA	Trip distance—work related	5 mi	1.88
			10 mi	0.94
			20 mi	0.47
		Trip distance—nonwork related	5 mi	2.02
			10 mi	1.02
			20 mi	0.51
Black and Towriss (*31*)	NA	Car trips to and from work		0.55
		All trips in sample		0.70
Asensio and Matas (*36*)	NA	NA		0.98
Noland and Polak (*37*)	NA	Commuting		1.27
Bates et al. (*38*)	NA	NA		1.1
Ghosh (*39*)	NA	NA		1.17
Yan (*40*)	NA	NA		1.47
Small et al. (*33*)	NA	NA		0.65
Bhat and Sardesai (*41*)	NA	NA		0.26
Hollander (*42*)	NA	NA		0.10
Tilahun and Levinson (*26*)	NA	NA		0.89
Carrion-Madera and Levinson (*30*)	NA	NA		0.91
Hensher (*43*)	198	Long distance (<3 h)		0.57
Small et al. (*33*)	5,630	Commute		3.5
Lam and Small (*44*)	332	Male		0.66
		Female		1.4
Small et al. (*23*)	1,155	Commute		0.91
Brownstone and Small (*22*)	601	Commute		0.4

home is less important that the arrival time at work. The reliability of personal errands or leisure trips is also expected to be less valuable than a morning commute, because it is expected the arrival time is much less important for these trips. Freight trips may have a very high value of reliability, especially when delivery logistics are based on just-in-time deliveries, and late arrivals can have an impact on production. A review of the value of reliability was conducted at the SHRP 2 Reliability Workshop on the Value of Travel Time Reliability and Cost–Benefit Analysis (*29*). An overview and meta-analysis on this topic completed in 2010 is provided by Carrion-Madera and Levinson (*30*).

As accounting for travel time reliability is a relatively new concept in transportation benefit–cost analyses of roadway improvements, agencies are less likely to have developed reliability values than travel time values specific to their roadways and drivers. The default value for the reliability ratio used in the Analysis Tool is 0.8, but agencies are encouraged to use values appropriate for their own state or metropolitan area, if available.

Costs of Crashes

Crash Cost Reduction Due to Congestion Reduction ($NReduction_{FI}$ and $NReduction_{PDO}$)

The crash cost reduction due to congestion reduction has been estimated based on the crash rate–traffic density relationships presented in Chapter 2 under Evolution of Research Approach for Traffic Operational Analysis and summarized in Equations 5.12 through 5.25.

Crash Cost Reduction Due to Direct Safety Benefits of Design Treatments (DSB_{FSI}, DSB_{MI}, and DSB_{PDO})

Some design treatments have direct safety benefits apart from their potential congestion reduction effects (i.e., they reduce crashes even when installed on uncongested facilities). Chapter 4 discusses assumptions that can be applied for various treatments, summarized as values of p_i in Table 4.8. Only treatments that eliminate crashes (Classes IIA and IIB) have these direct safety benefits. Other treatments that reduce crash incident duration or otherwise reduce crash consequences would not have such benefits because they do not reduce the number of crashes. Direct safety benefits may be used, if desired, to supplement the congestion-related effects on safety.

Crash Costs (C_{FSI}, C_{MI}, and C_{PDO})

Most highway agencies assign a cost savings to crashes reduced for each level of crash severity, based on either their own experience or on published values from the U.S. DOT or the National Safety Council. The benefit–cost analysis methodology developed by the research team uses the default values shown in Table 6.3, which were taken from recent U.S. DOT data (*9*) as adapted for use in *SafetyAnalyst* (*45*), but agencies are free to replace these values with values from other sources, such as the Insurance Institute for Highway Safety, or with their own agency's values, as appropriate.

Table 6.3. Default Values of Crash Costs by Severity Level

Severity Level	Cost Savings per Crash Reduced ($)
Fatal and severe injury	1,908,000[a]
Minor injury	51,000[b]
Property-damage only	4,000

[a] Weighted average crash cost based on costs of $5.8 million for fatal crashes and $402,000 for incapacitating-injury crashes.
[b] Weighted average crash cost based on costs of $80,000 for nonincapacitating-injury crashes and $42,000 for possible-injury crashes.

CHAPTER 7

Analysis Tool and Underlying Equations: Test for Reasonableness

Objective

The research team developed an analysis tool to implement the analytical procedures developed in this research. The purpose of the Analysis Tool is to allow highway agencies to analyze and compare the effectiveness of a range of design treatments at improving travel time reliability for a given highway segment. As part of a quality control review, the research team performed a series of sensitivity analyses using the tool to identify any errors and to assess the reasonableness of the results it provides to users. This exercise was useful in identifying inconsistencies in the Analysis Tool itself and in identifying inputs or default values that may cause the Analysis Tool to give unrealistic results.

Approach

Test scenarios were developed to represent realistic conditions for typical freeway sections; test scenarios for extreme conditions were also developed. Data representing these various sets of conditions were entered into the Analysis Tool. Using the default values for user-defined treatment-specific parameters, results were calculated by the Analysis Tool that predicted the delay savings and reliability measures for each scenario. The net present benefit of each scenario was calculated on the basis of the delay savings, safety benefits (direct and indirect), and reliability improvements. This quality control process was iterative: the Analysis Tool generated results for a set of scenarios, and the research team identified particular treatments or input variable combinations that gave unrealistic results. In these cases, the research team reconsidered the assumptions and rationale for choosing these default values and made changes as appropriate. In some cases, errors in the calculations were discovered and corrected.

The research team devised a two-pronged approach for testing the reasonableness of the Analysis Tool: a manual testing process and an automated procedure.

Members of the research team who were not involved in the construction of the Analysis Tool conducted manual testing of the tool. These research team members entered data into the Analysis Tool by hand, just as end users would, and recorded results in a separate document. This approach provided the opportunity for an additional check of user friendliness by users unfamiliar with the tool interface. The 16 scenarios shown in Table 7.1 were tested by using the manual method.

Because the manual testing was labor intensive, an automated procedure was developed to rerun the 16 scenarios listed in Table 7.1. By using the automated approach, the results of the 16 scenarios could be quickly plotted in various ways to identify additional or new unrealistic results that were not identified in the previous iteration.

Initial Results of Reasonableness Tests

The results of the manual testing were plotted for each of the 16 design treatments in the Analysis Tool. Figure 7.1 presents a plot of the results for crash investigation sites. For more information about the scenarios represented by each of the 16 bars in Figure 7.1, refer to Table 7.1.

Plots were also created showing all 16 design treatments applied to one scenario. This comparison was useful in identifying design treatments that appeared to yield unrealistically high or low benefits compared with other design treatments. For example, as Figure 7.2 shows, wildlife crash reductions are estimated to provide a very large net present benefit compared with the other design treatments. Although wildlife crash reduction treatments may be very beneficial in some areas, the research team concluded that this result was due to overestimation of treatment effectiveness at reducing crashes and underestimation of treatment implementation costs. Adjustments were subsequently made to the default parameters in the Analysis Tool and incorporated into future iterations of the automated testing. Treatment costs and default values related

Table 7.1. Scenarios Tested by Using the Manual Method

Scenario	No. of Lanes	ADT	No. of Incidents	Location
1	2	52,500	500	Orlando, Fla.
2	4	105,000	500	
3	2	30,000	500	
4	4	60,000	500	
5	2	52,500	100	
6	4	105,000	100	
7	2	30,000	100	
8	4	60,000	100	
9	2	52,500	500	Duluth, Minn.
10	4	105,000	500	
11	2	30,000	500	
12	4	60,000	500	
13	2	52,500	100	
14	4	105,000	100	
15	2	30,000	100	
16	4	60,000	100	

Note: ADT = average daily traffic.

to effectiveness (such as the number of crashes expected to be reduced by the design treatment) can be adjusted by the user to match local conditions.

Adjustments to Defaults

On the basis of the initial testing described above, the research team modified tool input default values in the following ways:

- Corrected an error identified in the net present value calculation.
- Corrected an error identified in the calculations for distributing crash totals to each hour of the day.
- Wildlife crash reduction
  - Reduced the default percentage reduction of property-damage-only, minor-injury crashes, and major-injury crashes (and other noncrash incidents) associated with this design treatment because for most freeway segments, animal–vehicle collisions make up a small proportion of total crashes (as the model is very sensitive to crash reductions, the team chose to err on the side of underestimating benefits with default values);
  - Refined initial values used for default installation cost on the basis of the best available information for wildlife crossing treatments; and

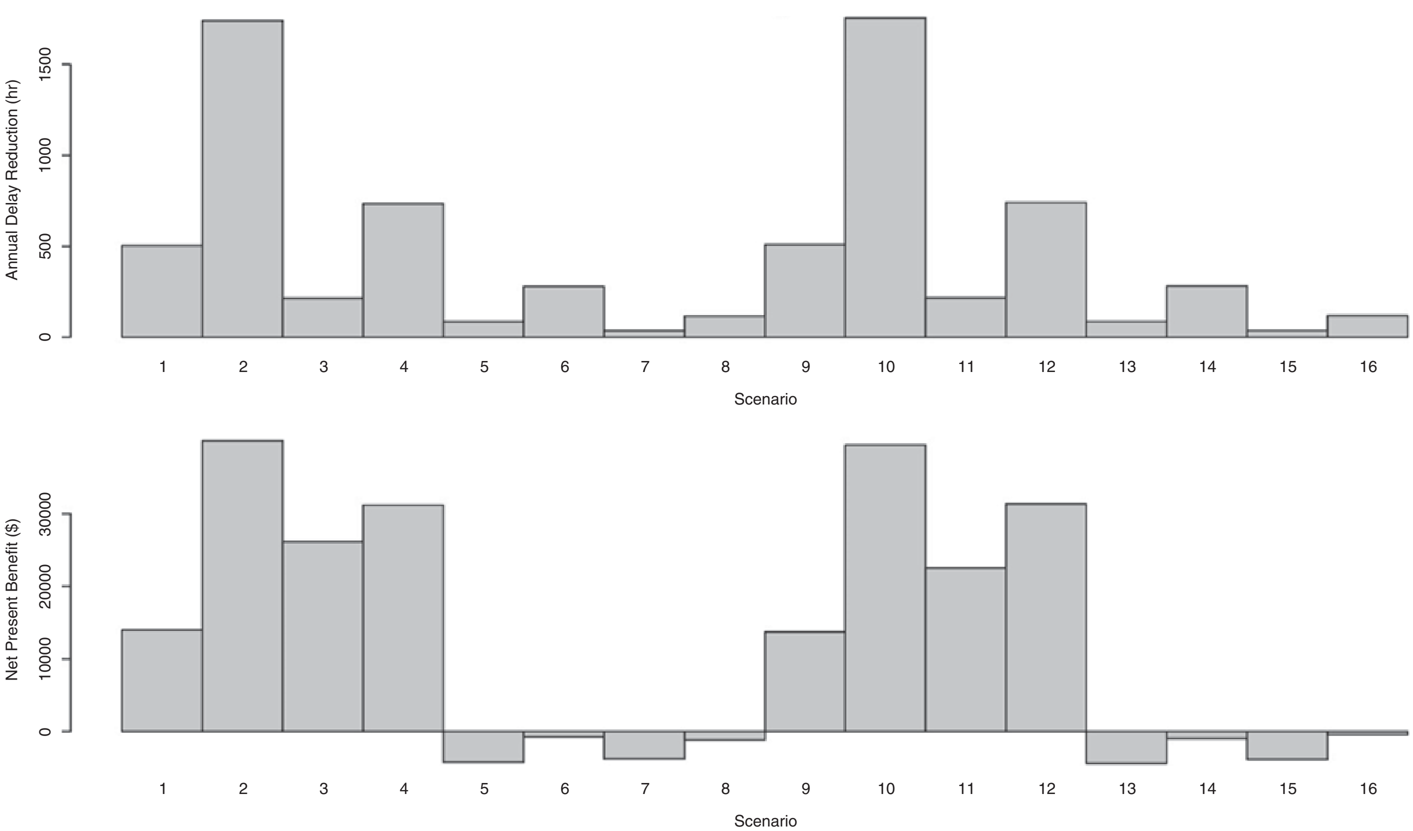


Figure 7.1. Initial results of manual testing for crash investigation sites.

 - Set default fatal crash reduction to 0% to err on the side of a conservative benefit estimate, given that most freeway segments will not experience many fatal animal–vehicle collisions.
- Snow fence
  - Refined the default installation cost on the basis of the best available information for the installation and maintenance cost of a typical snow fence; and
  - Set default fatal crash reduction to 0% to err on the side of a conservative benefit estimate, given that most freeway segments will not experience many snow-related fatal crashes that would be alleviated by a snow fence.
- Anti-icing systems
  - Refined the default installation cost on the basis of the best available information on the installation and maintenance costs for such systems; and
  - Set default fatal crash reduction to 0% to err on the side of a conservative benefit estimate, given that most freeway segments will not experience a significant amount of icy conditions.
- Drivable shoulder: reduced default shoulder capacity.
- Blowing sand: set default fatal crash reduction to 0% to err on the side of a conservative benefit estimate, given that most freeway segments will not experience a significant amount of blowing sand conditions.
- Output the benefit–cost ratio for each design treatment for each scenario and created graphics similar to the delay and net present benefit charts shown in Figures 7.1 and 7.2.

Although many default values for cost and crash reduction were altered during the validation process to produce conservative benefit estimates representing a typical site, analysts using the benefit–cost analysis procedures should change these defaults to better represent the specific characteristics of their site and planned treatment implementation.

Final Results of Reasonableness Tests

After implementing the changes to the Analysis Tool listed above, the automated procedure was used to generate plots showing delay, net present value improvements, and benefit–cost ratios for each of the 16 scenarios with each of the 16 design treatments. These plots are shown in Figures 7.3 through 7.5. The number above the bar in these three figures indicates the number of lanes (2 or 4); L and H indicate low and high crash counts, respectively. Table 7.2 summarizes the scenarios and codes presented in Figures 7.3 through 7.5.

(text continues on page 70)

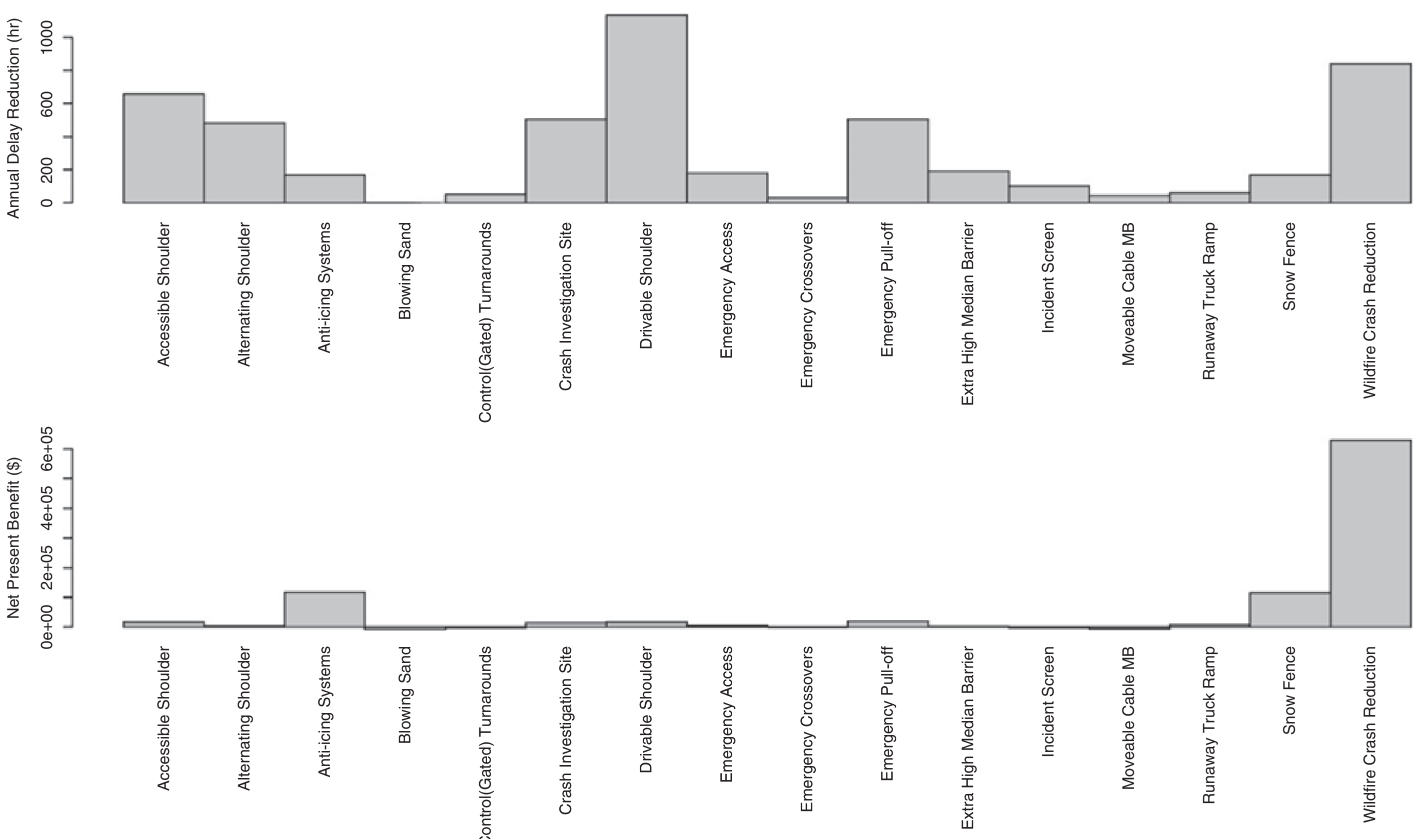
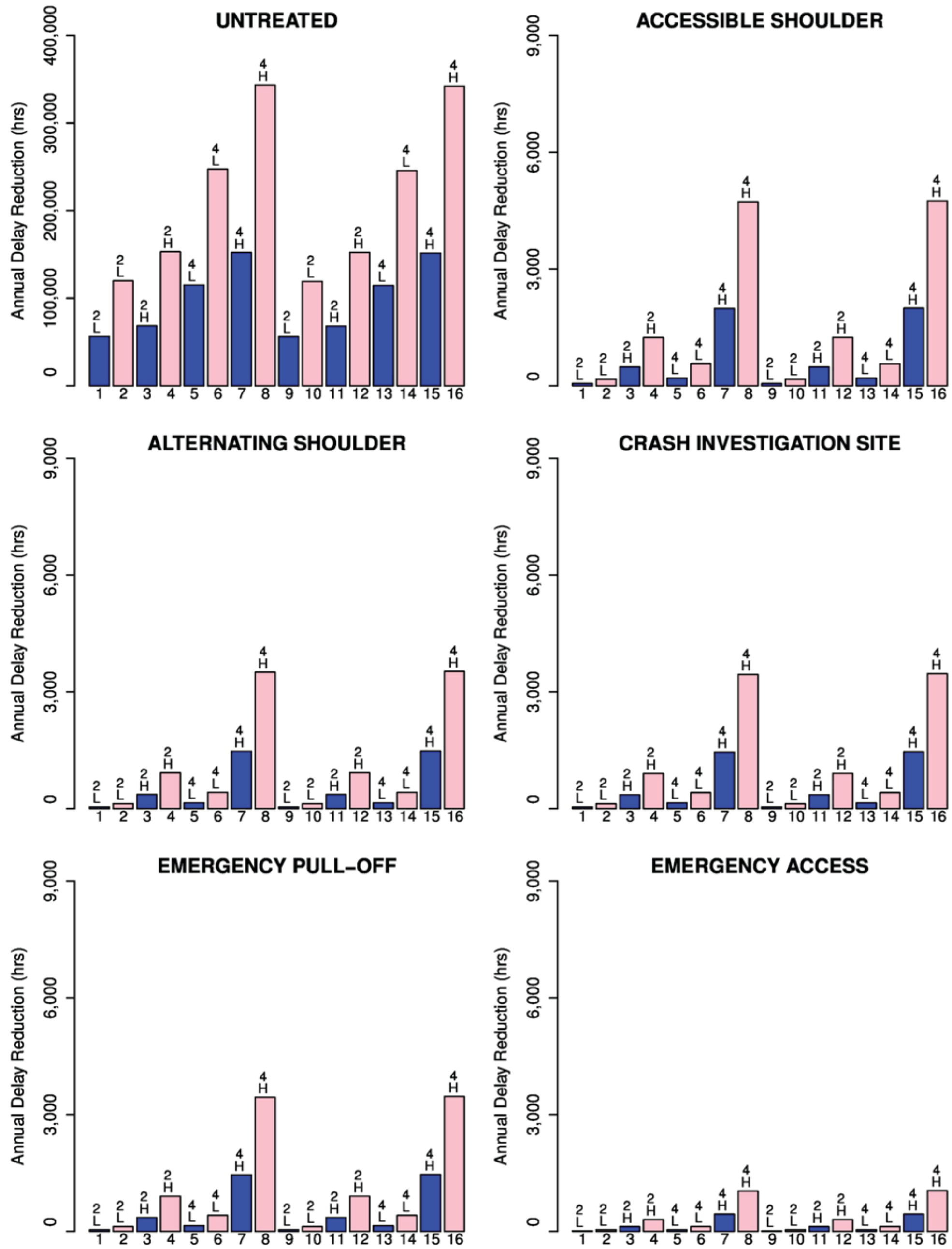


Figure 7.2. Initial results of manual testing: Scenario 1.



Note: Number above bar indicates number of lanes (2 or 4); L = low crash count; H = high crash count.

Figure 7.3. Delay reduction results.

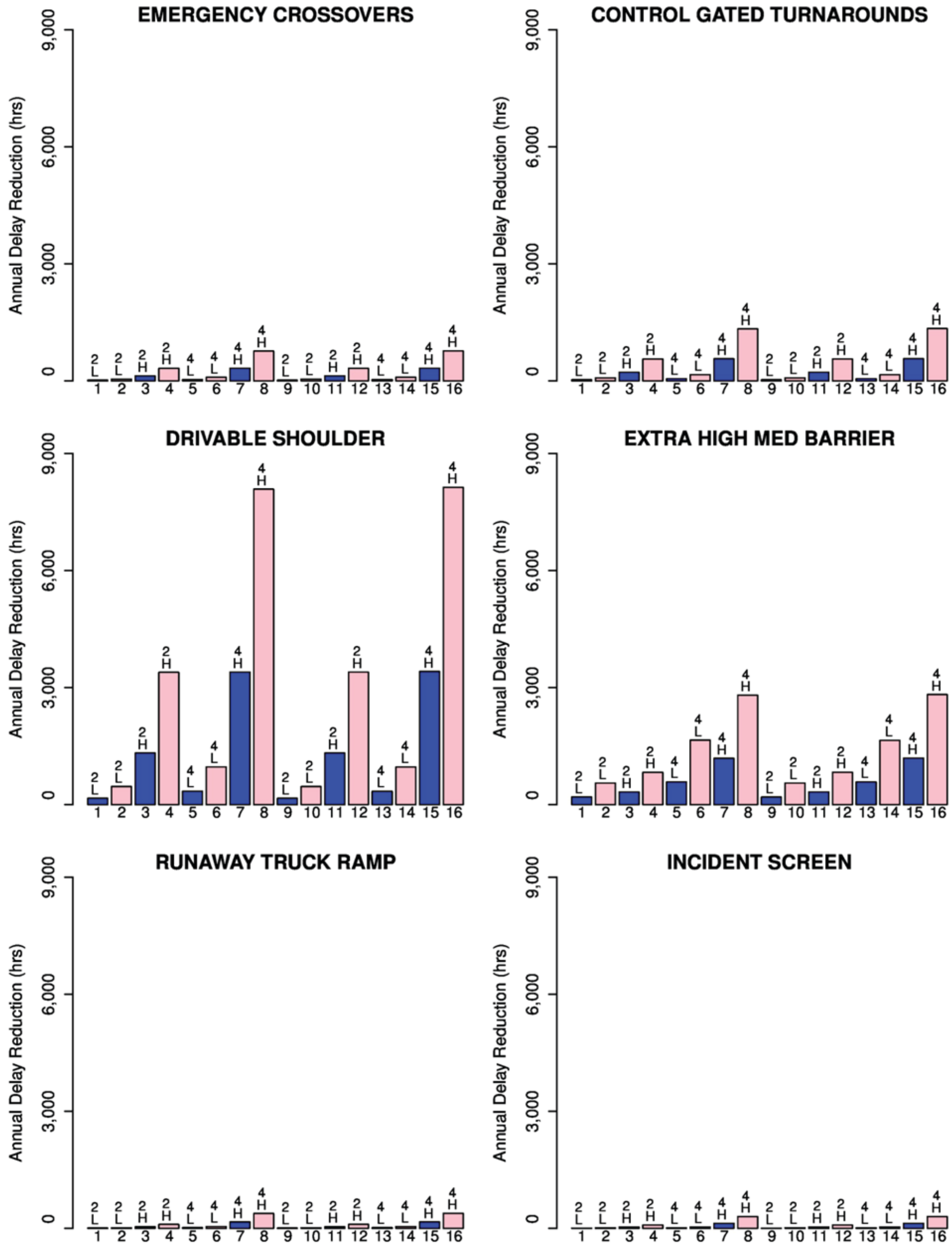


Figure 7.3. Delay reduction results. (Continued)

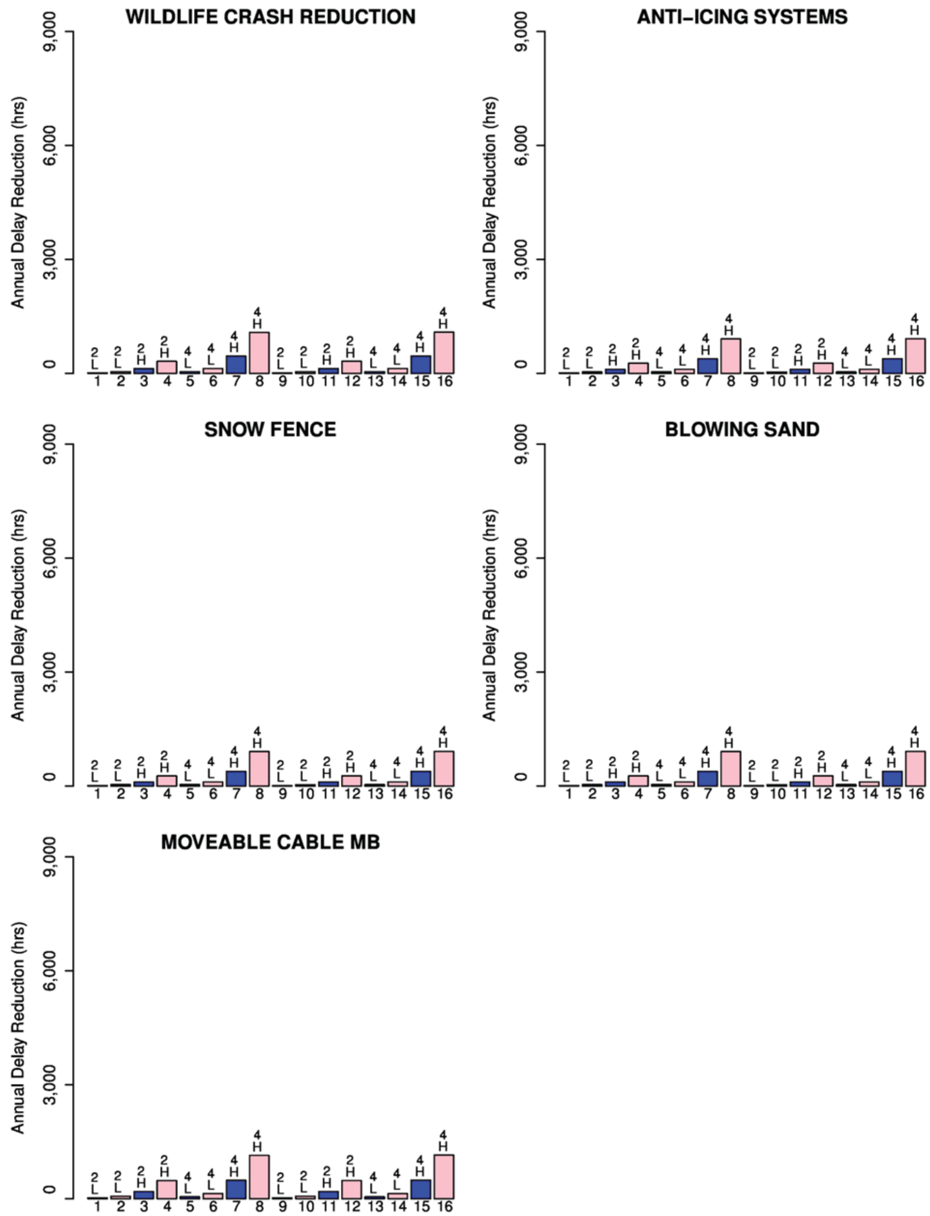
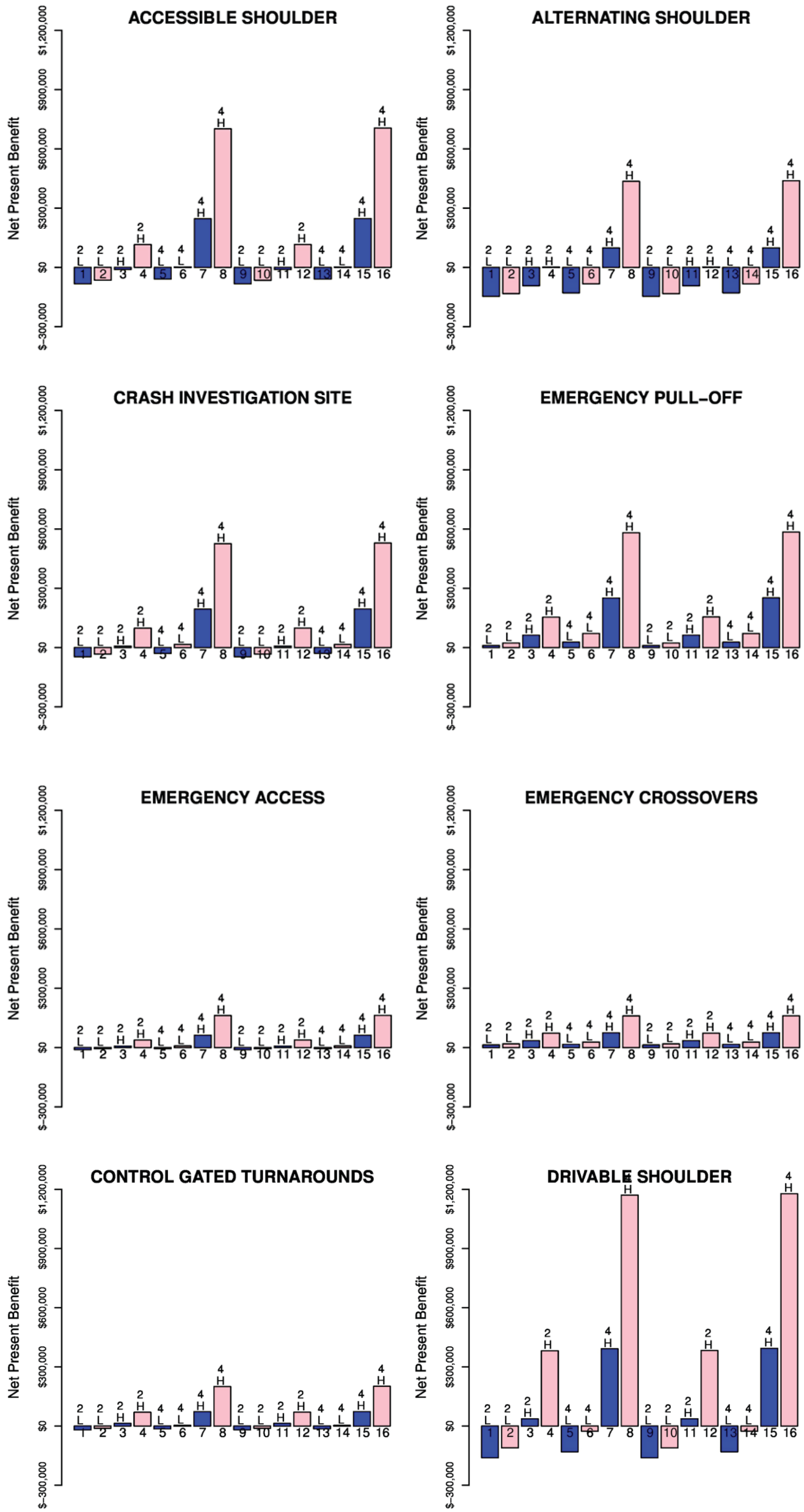


Figure 7.3. Delay reduction results. (Continued)



Note: Number above bar indicates number of lanes (2 or 4); L = low crash count; H = high crash count.

Figure 7.4. Net present benefit results.

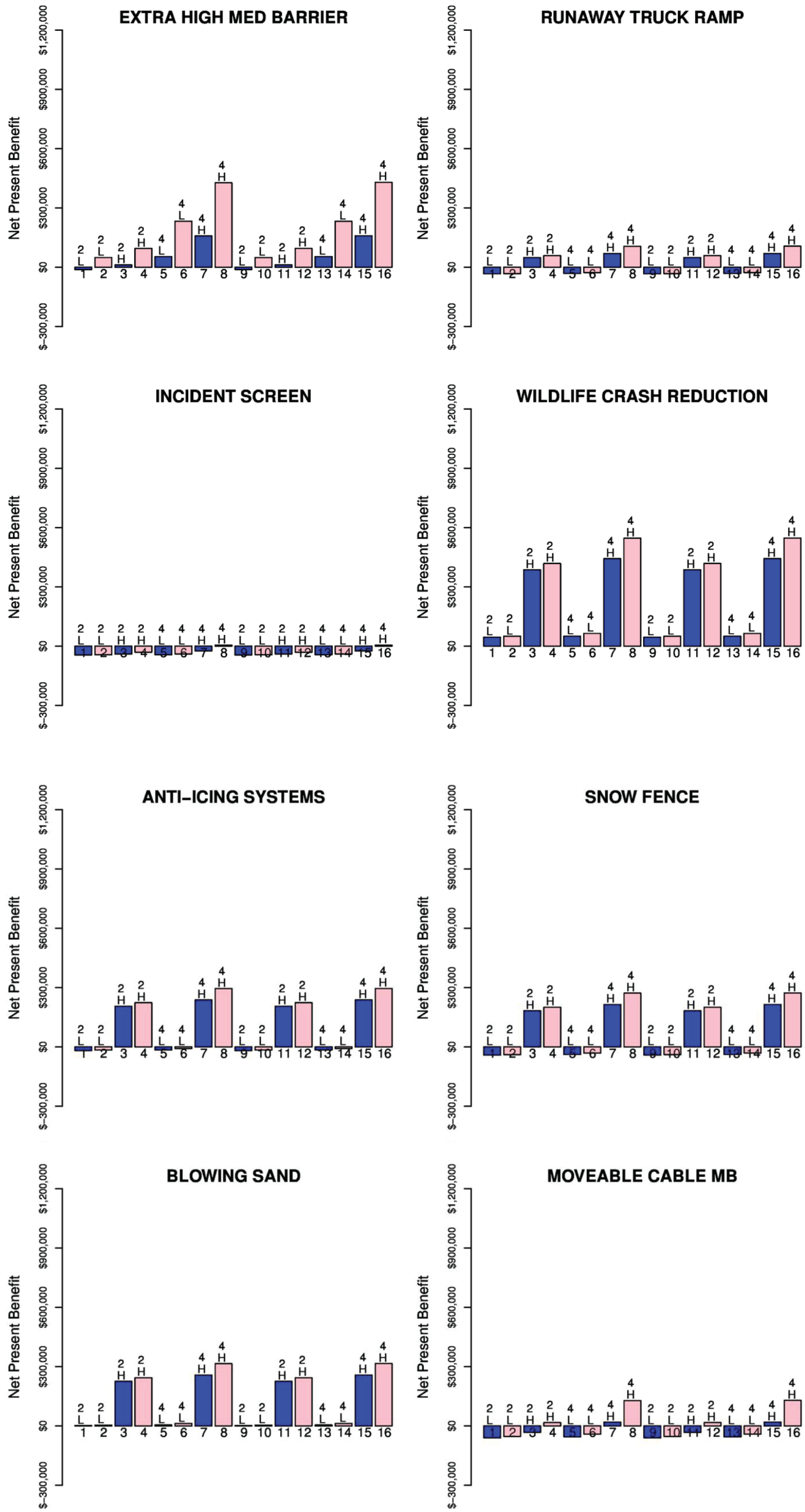
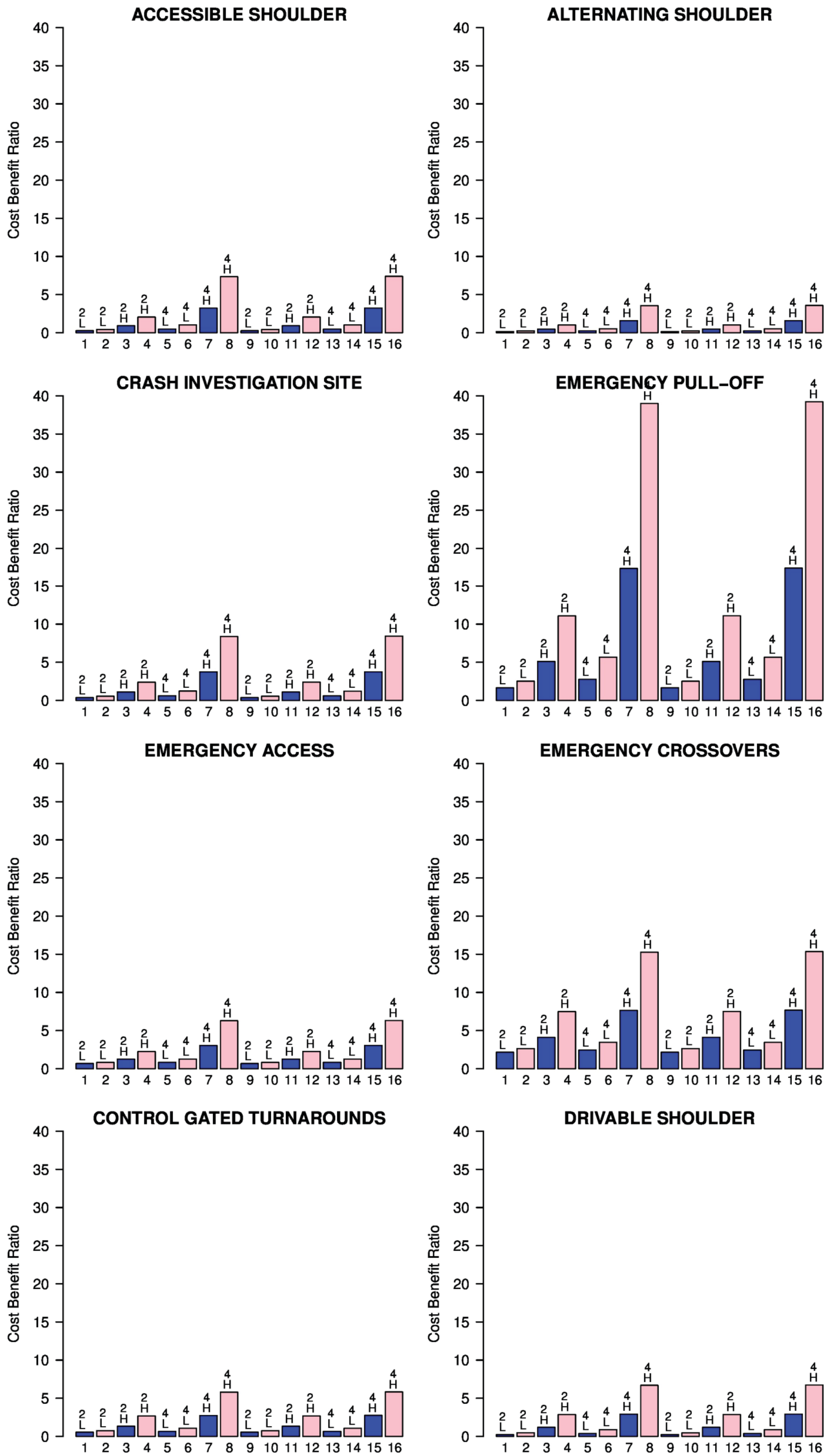
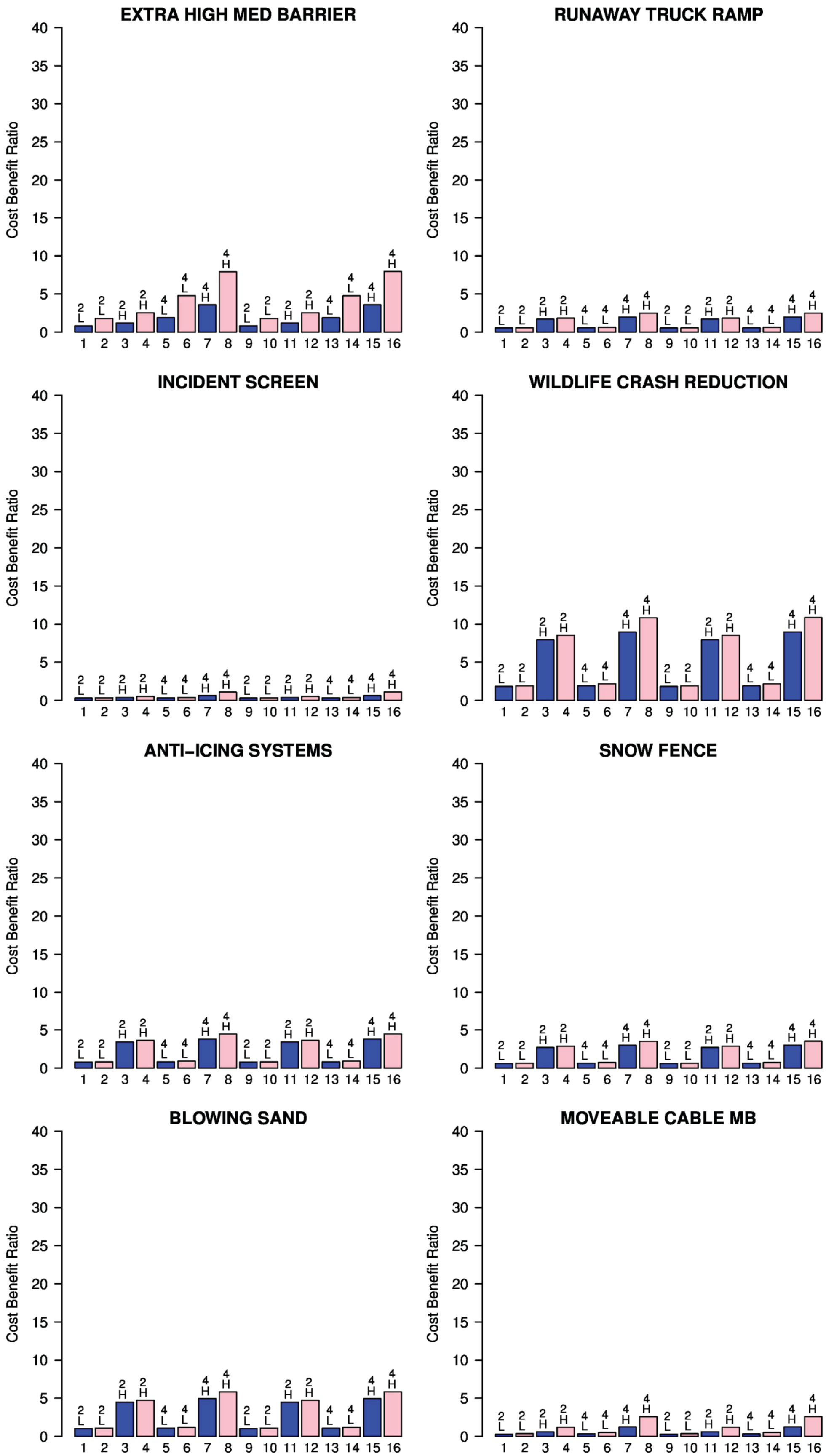


Figure 7.4. Net present benefit results. (Continued)



Note: Number above bar indicates number of lanes (2 or 4); L = low crash count; H = high crash count.

Figure 7.5. Benefit–cost ratio results.



Note: Number above bar indicates number of lanes (2 or 4); L = low crash count; H = high crash count.

Figure 7.5. Benefit–cost ratio results. (Continued)

Table 7.2. Scenarios and Codes Used in Figures 7.3 Through 7.5

Scenario	Location	No. of Lanes	Total Crash Count	Total Volume (ADT)	ADT Color Code
1	Orlando, Fla.	2	59	30,000	Light
2		2	59	52,500	Dark
3		2	295	30,000	Light
4		2	295	52,500	Dark
5		4	59	30,000	Light
6		4	59	52,500	Dark
7		4	295	30,000	Light
8		4	295	52,500	Dark
9	Duluth, Minn.	2	59	30,000	Light
10		2	59	52,500	Dark
11		2	295	30,000	Light
12		2	295	52,500	Dark
13		4	59	30,000	Light
14		4	59	52,500	Dark
15		4	295	30,000	Light
16		4	295	52,500	Dark

(*continued from page 62*)

Findings of Reasonableness Tests

The results of the reasonableness testing of the Analysis Tool and underlying equations led to the following conclusions:

- *Models are very sensitive to crash frequency.* The magnitude of treatment benefits is very sensitive to the annual number of crashes. A relatively small reduction in the annual crash total can result in a substantial increase in treatment benefits, particularly if the freeway section being analyzed experiences moderate to high congestion at some point during a typical weekday. This result makes sense because a reduction in crash frequency not only results in delay savings and reliability improvements, but also provides a direct savings of the cost of the crash itself.
- *Models are very sensitive to incident duration.* The duration of lane-blocking time for incidents has a dramatic impact on treatment benefit. By reducing incident clearance time or providing areas off the roadway for crash-involved or disabled vehicles (e.g., crash investigation sites), substantial delay reductions can be achieved.
- *Drivable shoulders provide high net present benefits.* Drivable shoulders were found to provide substantial benefits, especially as compared with other design treatments analyzed by the Analysis Tool (on the typical freeway sections analyzed using the 16 scenarios). On investigation of the default parameters of this design treatment, the assumptions and results appear to be reasonable.
- *Benefit–cost calculations are not sensitive to local weather conditions.* Weather conditions in Duluth, Minnesota, and Orlando, Florida, are substantially different. However, differences in net present benefits of design treatments applied in these two locations were negligent. Although rain and snowfall affect the TTI curves for both treated and untreated conditions, they appear to affect these curves proportionally, so that the difference between the treated and untreated curves does not change substantially.

CHAPTER 8

Conclusions and Recommendations

This chapter presents both general conclusions of the research and recommendations for the implementation of the research results. The conclusions are discussed as basic summaries of what was learned through literature reviews, interviews with highway agencies, careful examination of research methods and findings from SHRP 2 Project L03 (a key foundation of this research), and methods that were developed by this research team to meet the project objectives. The recommendations presented in this chapter are geared toward highway agency decision makers, including planners, traffic and operational engineers, and managers, who seek to maximize the potential operational benefits of their freeway design decisions within their resource constraints.

Conclusions of the Research

Geometric Design Treatments and Nonrecurrent Congestion

The research team found that highway agencies tend to address recurrent congestion issues with infrastructure treatments and nonrecurrent congestion with intelligent transportation system treatments. That is, daily demand peaks that cause peak hour congestion are often treated by adding base capacity. Congestion caused by incidents, special events, work zones, and other infrequent and unpredictable events are typically addressed by providing travelers with real-time information through traffic management centers. These centers monitor freeways and post information about travel time, lane blockages, and alternate routes to drivers in real time via radio, websites, and message boards. Geometric design treatments that address base capacity issues have been investigated and evaluated thoroughly in the literature, and more recently, operations-based treatments such as real-time traveler information and motorist-assist patrols have been evaluated for their effectiveness at alleviating nonrecurrent congestion. However, the use of geometric design treatments to help reduce nonrecurrent congestion is not well documented in the literature.

Through interviews with highway agencies, the research team identified instances of agencies using design elements to help manage nonrecurrent congestion; however, in most cases these treatments had not been designed specifically for this purpose. Instead, treatments designed to manage recurrent congestion were manipulated to apply to nonrecurrent congestion events, frequently in an ad hoc fashion. When major incidents occurred, agencies would use whatever tools were at their disposal to minimize the disruption to traffic. Typically, the facility was not "designed" to function as a treatment for nonrecurrent congestion, and usually there was no policy in place to implement the treatment under certain defined conditions. For example, some agencies will open a shoulder as a driving lane to bypass an incident causing congestion, even though having this option available was not specifically considered during the design of the shoulder, and the decision to implement shoulder driving is made by on-site responders, rather than defined in policy.

This research fills an important gap in the literature by documenting the benefits of using design treatments to reduce nonrecurrent congestion and by encouraging the consideration of these benefits during the planning and design phases of highway projects. By more accurately predicting the benefits of these types of treatments, decision makers are better informed of the available options for addressing nonrecurrent congestion, and greater benefits to the traveling public can be achieved.

Relationship Between Nonrecurrent Congestion and Reliability

The literature contains a great deal of research on transportation reliability, but there is no consensus on the definition of reliability for roadway segments. Reliability is often discussed in the literature in terms of trip reliability (sometimes for a specific subset of vehicles, such as freight deliveries or commuters), measuring the percentage of on-time trips or the variation between actual trip time and ideal trip time. This research explores the reliability of a specific segment of roadway and includes the travel times of all vehicles traveling across the segment. The segment travel time is only one part of each of the various trips made by the drivers on that segment, so little can be known about the reliability of any driver's trip. However, this analysis can help highway agencies evaluate how well a certain segment of roadway is operating and if it is contributing to trip delay and reliability issues for the drivers using it. This approach makes sense when evaluating geometric design treatments that are applied at specific locations on the roadway. This measure of reliability can be used to evaluate how improvements to a section of roadway reduce delay and improve reliability along that section.

This research adopted the definition of segment reliability used in SHRP 2 Project L03. For the purposes of the present research, reliability is a measure of the variation in travel times across the segment over a long period of time (here, 1 year). Reliability describes only one characteristic of freeway operations: the predictability of travel times. Delay is another characteristic of freeway operations. Both recurrent congestion (resulting from inadequate base capacity for daily demand) and nonrecurrent congestion (resulting from crashes, incidents, weather, work zones, and special events) cause delay, and roadway users incur costs from either type of delay. Recurrent congestion alone is generally predictable, and therefore familiar drivers can estimate their travel time accurately, factoring in the expected amount of delay, when only recurrent congestion is present. However, roadway users incur additional costs when they experience nonrecurrent congestion and their travel times vary from one day to the next. On roadway segments with substantial nonrecurrent congestion, drivers must plan for a longer-than-average trip every day to accommodate the possibility of unexpected congestion, which leads to wasted time. This travel time variability can be described in terms of reliability. Reliability is evaluated separately for each hour of the day, so that the analyst may find a road to be highly reliable during off-peak hours and not very reliable during peak hours.

The research team found that events that cause nonrecurrent congestion have a much bigger impact on reliability during hours of recurrent congestion (i.e., hours with high delay). That is, a crash or work zone will have a bigger impact on travel time when traffic is already congested. For this reason, treatments that reduce recurrent congestion will have a positive impact on reliability. In addition, design treatments that address nonrecurrent congestion will have greater benefit on roadways that experience congestion or regularly operate with a demand that approaches capacity (where even a minor disturbance could cause congestion).

The primary causes of nonrecurrent congestion on freeways are traffic crashes and other incidents, special events, work zones, weather, demand surges, and sometimes traffic control devices (such as malfunctioning ramp meters). These events cause congestion either by reducing the effective capacity of the roadway or by increasing demand. For example, snow storms often reduce the capacity of a four-lane freeway segment to two lanes, and crashes often block one or more lanes. Special events, such as sporting events and concerts, can substantially increase the demand on a freeway segment before and after the event. Design treatments that can help increase capacity (or decrease the lost capacity) or decrease demand will help to reduce the impact of these events on congestion, and therefore improve reliability.

Reliability can be a good measure of the impact of nonrecurrent congestion on the operation of a roadway, especially for roadways that experience incidents that cause nonrecurrent congestion fairly regularly. However, very infrequent major incidents that last for several hours and block several lanes of traffic or shut down a road entirely are not well captured in a reliability measure. Because reliability captures the day-to-day variation of travel times on a segment of roadway, a major incident occurring on a roadway that rarely experiences any congestion (either because incidents are infrequent or because traffic demand is low enough that incidents have a very minor impact) may not have much of an impact on reliability. If the roadway operates smoothly 364 days of the year, but is shut down for 1 day, it is highly reliable, despite having serious impacts on the motorists trying to use the roadway on that particular day. And because reliability is measured individually for certain hours of the day, the impact of a catastrophic event is typically spread over several hours. So, although there are treatments that may help alleviate the consequences of major catastrophic incidents (such as using a median opening to allow trapped traffic to turn around), the benefits of these treatments may be more appropriately measured in terms of delay reduction for individual incidents rather than in terms of reliability improvement.

Evaluating Treatment Impacts on Reliability

Project L03, which preceded this research effort, developed models for predicting a travel time index (TTI) at various percentiles. The input variables to the models were a measure of lane hours lost due to incidents and work zones, the number of hours during the year during which more than a trace amount of rain fell, and the critical demand-to-capacity (*d/c*) ratio for the roadway segment, all during the particular timeslice being evaluated (e.g., 5:00 to 6:00 p.m.). As explained in detail in Chapter 1, these TTI percentiles can be used to estimate a cumulative distribution of TTIs, from which many observations and measurements can be made.

As part of the Project L07 research effort, the research team improved on these models in two important ways. First, the Project L03 models were found to be based on data from cities that did not experience significant snowfall, so this research incorporated a snowfall variable in addition to the rainfall variable in the models. Second, the Project L03 models were developed for peak hours in large metropolitan areas. This research developed additional models to be used for facilities or hours of the day (or both) with lower *d/c* ratios. Models were needed that could be applied to all 24 h of the day so that the full benefit of treatments that could potentially be used during any hour of the day could be accounted for. The resulting set of models estimates the distribution of TTIs for a given freeway segment for each hour of the day by using four input variables: rainfall, snowfall, *d/c* ratio, and lane hours lost.

As explained in Chapter 1, the shape of the cumulative TTI curve provides a great deal of information about delay and reliability. A curve with a nearly vertical line at TTI = 1.0 indicates that almost every trip on that segment is made at free-flow speed, which means that the roadway is reliable and that drivers experience very little delay. A hypothetical curve with a steeply vertical line at a higher TTI would indicate reliability (very little variance in TTI), but that most drivers experience delay because their trip takes longer than it would at free-flow speed. A curve with a strong "lean forward" indicates a high variability in TTI and, therefore, lower reliability.

To measure the impact that a specific design treatment has on reliability, the research team developed a method of measuring the difference between a TTI curve for a roadway in an untreated condition and a TTI curve for the treated condition. To develop the curve for the treated condition, the impact of the design treatment must be described in terms of the four model input variables. In general, most treatments have an effect on the lane hours lost variable by minimizing the number of incidents that occur, reducing the time that lanes are closed or blocked by traffic incidents or work zones, or providing extra capacity during events that close lanes. Hours of rain or snowfall cannot be affected by design treatments, but their impacts on lane capacity can be affected by treatments such as snow fences and anti-icing treatments. Some treatments also affect the *d/c* ratio. Once the impacts on these variables are determined for a given treatment, the delay reduction and improvement in reliability can be measured by analyzing the difference between the two TTI curves.

The degree to which treatments affect the lane hours lost or *d/c* ratio input variables is highly dependent on site-specific characteristics, as well as implementation and policy decisions. For example, a jurisdiction that provides easily accessible, well-signed crash investigation sites and enforces a policy that all crashes must be moved to one of them if possible will see a greater impact on the lane hours lost variable than an agency that implements only a few sites that are hidden from view of the public and that law enforcement rarely uses. Therefore, it is only possible to estimate the potential impact of a design treatment when information is known about the likelihood and frequency with which it will be used.

Relationship Between Nonrecurrent Congestion and Safety

This research explored the relationship between congestion and safety—specifically the relationship between level of service (LOS) and crash frequency—and developed a mathematical model to quantify the increase in crash frequency at all severity levels as LOS worsens. Crash frequency is lowest at LOS B and into LOS C, but then begins increasing through LOS D, E, and F. This relationship indicates that if LOS can be improved (by implementing design treatments that decrease congestion) in the range from LOS C to LOS F, then crash frequency will fall. Therefore, treatments that reduce congestion also improve safety.

Benefit–Cost Analysis of Design Treatments for Nonrecurrent Congestion

One of the objectives of this research was to conduct a benefit–cost evaluation for the various design treatments that were evaluated. Because both the benefits and the implementation and maintenance costs of the treatments are dependent on existing site characteristics, specific implementation plans, and accompanying policies for use, a spreadsheet-based analysis tool was developed to allow agencies to estimate the potential benefit of a specific implementation of a treatment in a specific location. This tool also allows agencies to compare the benefits of various treatments as they might be implemented in a given location.

In the tool, both construction and annual maintenance costs are entirely user defined. Initially, the research team considered providing default values for treatment costs. However, the team received feedback from potential tool users that agencies can easily estimate these costs, and that as construction and materials costs vary greatly from location to location, as well as over time, any defaults provided would likely be inappropriate for many users.

To calculate treatment benefits, three main components are considered: delay savings, reliability improvement, and safety improvement. Using the untreated (base condition) TTI curve and the treated (after treatment implementation) TTI curve, a reduction in delay due to treatment implementation can be calculated. This measurement is in terms of vehicle hours, which is converted to dollars by assigning a monetary value to travel time. Many agencies have a default value that is typically used to convert delay hours to economic cost in dollars. A change in reliability can also be determined on the basis of the shift in TTI cumulative curves from untreated to treated. In this project, reliability was quantified as the standard deviation of the travel time distribution, converted into units of hours. There is no consensus in the literature on how this measure should be valued in economic terms, but one common method is to use a reliability ratio. A reliability ratio is the ratio of the value of reliability to the value of time. By defining this ratio as a fixed number, the value assigned to reliability is always a multiple of the value of time. Just as the value of time may vary from one user group to the next (such as freight or peak hour commuters), so too can the reliability ratio vary from one group to the next. The research team defined the reliability ratio to be 0.8 for all travelers at all times of day in this research, which fell within the range of most values presented in the literature.

The results of this research provide a method for incorporating both the economic savings due to delay reduction and the economic savings due to reliability improvement for a design treatment over its life cycle. Treatments commonly used to address recurrent congestion can be analyzed using the approach developed in this research, which takes into account not only the delay improvements associated with the treatment, but the potential improvements to reliability, as well. Taking these benefits into account results in a more accurate valuation of a treatment's net present benefit and benefit–cost ratio. In addition, agencies considering removing roadway features beneficial to nonrecurrent congestion in order to alleviate recurrent congestion (such as by converting a shoulder to a driving lane) can use the methods presented in this report and the Analysis Tool to calculate the expected increase in nonrecurrent congestion and decrease in reliability that might be expected due to the change and compare these costs with the benefits achieved for recurrent congestion by adding additional capacity.

Recommendations for Implementation of Research Results and Future Research Needs

On the basis of the conclusions of the research effort described above, the L07 research team recommends the following:

- Reliability is an important measure of highway operations and has a value beyond delay savings. Design choices should be evaluated for the full range of benefits they may provide. Even design elements aimed at reducing recurrent congestion may affect nonrecurrent congestion and reliability.
- Improving reliability should be a goal for all highway design projects in the planning phase. Often, designs can be altered slightly to serve as or accommodate nonrecurrent congestion treatments at a minimal or negligible cost. Considering reliability impacts in the planning process will help maximize treatment benefits while minimizing implementation costs.
- Methods and procedures documented in this report and applied in the Analysis Tool should be adjusted to reflect specific local conditions as much as possible by replacing default values with local information. The impact a given treatment may have will be highly dependent on the specific site characteristics, implementation choices, and policies governing treatment use.
- Reliability should be considered not only when planning for the design of new facilities or major reconstruction, but when highways are being reconstructed to add capacity for recurrent congestion concerns. Although reducing recurrent congestion often reduces nonrecurrent congestion, this positive benefit can be negated when storage areas for vehicles involved in crashes or other incidents are removed. In these cases, lane-blocking time for a crash-involved vehicle may increase substantially, making the roadway significantly less reliable despite the additional capacity. The procedures and Analysis Tool developed in this project allow decision makers to weigh the costs of decreased reliability against the estimated costs of delay reduction from the capacity increase.

Potential research needs related to reliability analysis for nonrecurrent congestion and potential enhancements to the tool include the following:

- Developing the capability for the tool to import data from, or export data to, other software packages or databases to promote more efficient data analysis and reduce redundant data entry.
- Adding calibration and comparison features for users who have detailed TTI data for existing conditions.
- Developing methodologies for considering multiple treatments applied simultaneously.
- Extending the tool to allow analysis of facilities and corridors, not just segments.
- Improving the file and scenario management capabilities of the tool to make analysis of multiple sites easier.
- Expanding the tool to explicitly compare nondesign (operational or technology) treatments and recurrent congestion enhancements to the base (no treatment) case.
- Expanding the tool to explicitly evaluate the operational and safety effects of removing a treatment (e.g., converting a drivable shoulder to a driving lane).
- Incorporating into the benefit–cost methodology and Analysis Tool additional treatment benefits, such as fuel and other vehicle operating costs savings and emissions reduction.
- Including the capability to specify traffic growth over the design life of the treatment in the benefit–cost methodology and the Analysis Tool.
- Refining the safety–congestion relationship with data from additional cities or regions.

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APPENDIX A

Background Information for Model Development

The reliability models developed in SHRP 2 Project L03 served as a starting point in Project L07 for evaluating the effectiveness of design treatments in reducing nonrecurrent congestion and improving travel time reliability. However, although these models included a variable ($R_{0.05''}$) to account for rainfall, they did not account for snow conditions. Furthermore, the Project L03 models were more applicable to congested conditions and were not developed for the full range of demand-to-capacity (*d/c*) ratios. To address these and other issues, SHRP 2 approved an extension of Project L07 to further develop and refine the analytical framework and the spreadsheet-based Analysis Tool that were developed in the research.

This appendix describes in detail the work conducted to (1) further develop the models to address the effects of snow and ice on the traffic operational effectiveness of design treatments and (2) develop reliability models for time periods with $d/c < 0.8$.

Further Develop Models to Address Effects of Snow and Ice

The objective of this effort was to develop a method for incorporating consideration of snow and ice into the reliability models used to assess design treatments. The research team used existing traffic operational data from the Minneapolis–St. Paul, Minnesota, metropolitan area to quantify the relative effects of snow and rain on travel time reliability and incorporate an explicit snow-and-ice term into the reliability models. Lookup tables for the annual number of hours with snowfall above a threshold, analogous to those already developed for rainfall in Project L07, were developed for all U.S. weather stations that experience snowfall.

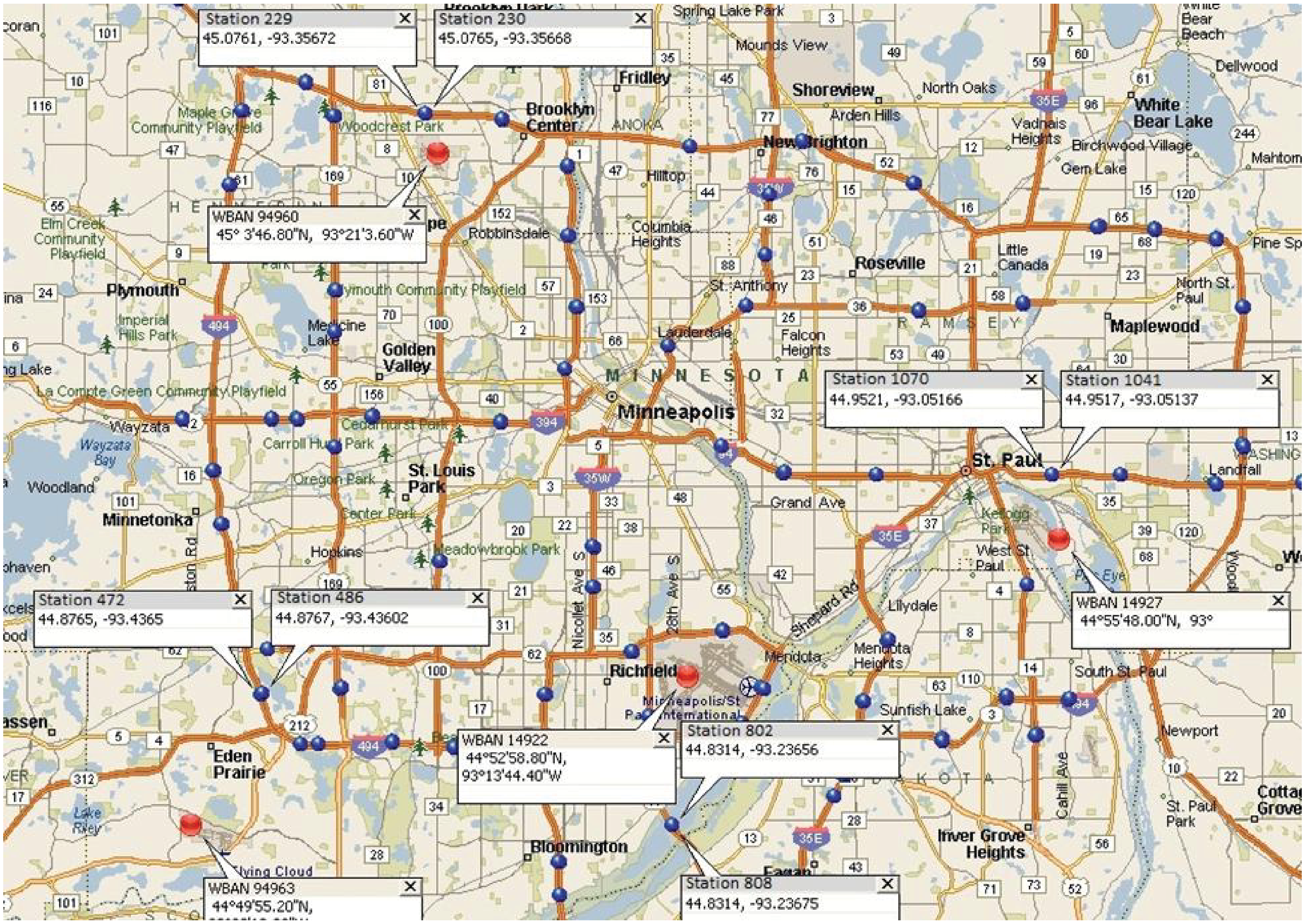
Project L03 accounted for the effect of rainfall on travel time reliability by incorporating a rainfall term ($R_{0.05''}$) into the reliability models. $R_{0.05''}$ is defined as the number of times (during a given time-slice, e.g., 8:00 to 9:00 a.m.) during a year that hourly rainfall is greater than or equal to 0.05 in. The threshold of 0.05 in. was determined, in Project L03, to be the amount of rainfall that begins to have a noticeable effect on vehicle speeds.

One of the first steps in this effort was to determine a similar threshold for snowfall. That is, what is the minimum amount of snowfall that begins to noticeably affect vehicle speeds? To determine this threshold, a database was assembled using weather data from the National Weather Service. Four pairs of freeway segments were identified in the Minneapolis–St. Paul area, with each pair corresponding to one of four weather stations. Figure A.1 shows the eight freeway segments and the corresponding four weather stations.

Speed and volume data for these freeway segments were already available from other analyses in Project L07. Each 5-min record included an average per lane speed and a per lane volume. To determine the minimum snowfall rate that has an effect on travel speeds, the data were filtered in several ways. First, hours with traffic volumes greater than 1,200 vehicles/h were excluded, because in these cases, congestion may contribute to a decrease in speed. Hours between sunset and sunrise were also excluded, because darkness may also contribute to speed reductions.

A mean speed value was plotted for each hour according to the recorded snowfall amount that occurred during the hour. As shown in Figure A.2, the majority of hours had no snowfall (0.00 in.). The mean speed of all "no precipitation" hours in the database was 67.0 mph. As the figure shows, there was a noticeable decrease in speed for snowfall amounts of as little as 0.01 in.; the mean speed of all hours with 0.01 in. of snowfall was 60.2 mph. The magnitude of the speed reduction effect appears to increase with increasing rates of snowfall until approximately 0.05 in. of snowfall per hour, at which point the snowfall effect remains fairly constant.

On the basis of this analysis, the research team concluded that the appropriate snow term to be used in the reliability



Source: © Microsoft Streets and Trips.

Figure A.1. Freeway stations used in Task IV-1 analysis.

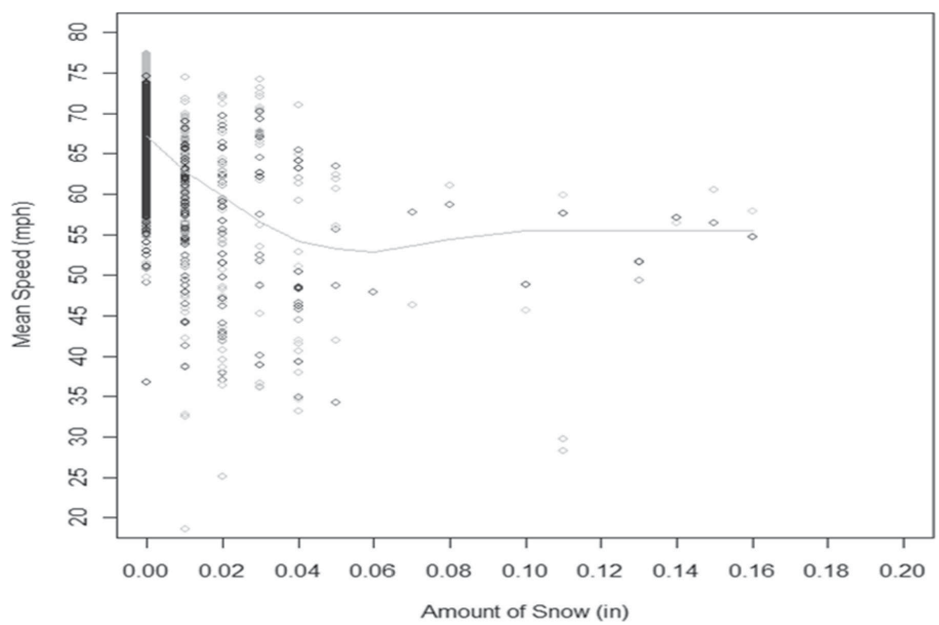


Figure A.2. Mean hourly speeds by hourly snowfall amount.

models was Snow01. This value is defined as the number of hours per year during a particular time-slice when snowfall exceeds 0.01 in.

Having defined the snow variable to be used in the reliability models, the research team developed a speed distribution for each hour by using average per lane speeds and per lane volumes from the raw data. The speed distribution for each hour was then assigned to one of three categories: no precipitation (NP), rain above 0.05 in. per hour, or snow above 0.01 in. per hour.

The 5th percentile speed was calculated for each hour at each freeway section. (The 5th percentile speed corresponds to the 95th percentile travel time index [TTI].) An average of the 5th percentile speeds were calculated for the NP hours, the rain hours, and the snow hours. The three average values were then compared. As expected, the average 5th percentile speed for NP hours was greater than the average 5th percentile speed for rain hours, which was greater than the average 5th percentile speed for snow hours. The following discussion describes how the results of the observed data analysis were incorporated into the L03 equations.

TTI for Hours with No Precipitation

The peak hour reliability model from Project L03 can be represented by Equation A.1:

$$\mathrm{TTI}_{n\%} = e^{(j_n \mathrm{dc}_{\mathrm{crit}} + k_n \mathrm{LHL} + l_n R_{0.05''})} \tag{A.1}$$

where

$\mathrm{TTI}_{n\%}$ = nth percentile TTI;

$\mathrm{dc}_{\mathrm{crit}}$ = critical d/c ratio within the time-slice of interest (e.g., 7:00 to 8:00 a.m.);

LHL = annual lane hours lost due to incidents and work zones that occur within the time-slice of interest (e.g., 7:00 to 8:00 a.m.);

$R_{0.05''}$ = hours in the year with rainfall ≥0.05 in. that occur within the time-slice of interest (e.g., 7:00 to 8:00 a.m.); and

j_n, k_n, l_n = coefficients that correspond to the nth percentile TTI.

Table A.1 lists the coefficients that correspond to the NP TTI percentiles under consideration.

The 95th percentile equation is the only percentile from the Project L03 peak hour model to have a nonzero coefficient for the rain variable. Therefore, the research team used the L03 peak hour model without the rain variable to develop a TTI for no precipitation (NPTTI), as shown by Equation A.2:

$$\mathrm{NPTTI}_{n\%} = e^{(j_n \mathrm{dc}_{\mathrm{crit}} + k_n \mathrm{LHL})} \tag{A.2}$$

where $\mathrm{NPTTI}_{n\%}$ is the nth percentile no precipitation TTI, and the other variables are as defined for Equation A.1.

Table A.1. Coefficients Corresponding to nth Percentile TTI: No Precipitation

TTI Percentile	j_n	k_n	l_n
10	0.07643	0.00405	0.00000
50	0.29097	0.01380	0.00000
80	0.52013	0.01544	0.00000
95	0.63071	0.01219	0.04744[a]
99	1.13062	0.01242	0.00000

[a]The 95th percentile equation is the only one with a rain variable.

TTI for Hours with Rain ≥0.05 in.

For each of the five TTI percentiles, speed data from those hours with rain ≥0.05 in. (RTTI_n) were compared with speed data from those hours with no precipitation, and the following regression equation (Equation A.3) was developed:

$$\mathrm{RSpeed}_n = m * \mathrm{NPSpeed}_n + b \tag{A.3}$$

where

RSpeed_n = average nth percentile speed for hours with rainfall ≥0.05 in.;

$\mathrm{NPSpeed}_n$ = average nth percentile speed for hours with no precipitation; and

m, b = coefficients for nth percentile TTI.

$\mathrm{NPSpeed}_n$ is calculated using the relationship shown by Equation A.4:

$$\mathrm{NPSpeed}_n = \frac{\mathrm{FFS}}{\mathrm{NPTTI}_n} \tag{A.4}$$

where FFS is free-flow speed.

Table A.2 lists the coefficients that correspond to the five TTI percentiles (with rain) under consideration.

The variable RSpeed_n can be converted back into a TTI for rain (RTTI) by using Equation A.5:

$$\mathrm{RTTI}_n = \frac{\mathrm{FFS}}{\mathrm{RSpeed}_n} \tag{A.5}$$

Table A.2. Coefficients Corresponding to nth Percentile TTI for Rain

TTI Percentile	*m*	*b*
10	1.364	–28.34
50	0.966	–6.74
80	0.630	6.89
95	0.639	5.04
99	0.607	5.27

TTI for Hours with Snow ≥0.01 in.

For each of the five TTI percentiles, speed data from those hours with snow ≥0.01 in. ($STTI_n$) were compared with speed data from those hours with no precipitation, and the following regression equation (Equation A.6) was developed:

$$SSpeed_n = m * NPSpeed_n + b \tag{A.6}$$

where

$SSpeed_n$ = average nth percentile speed for hours with snow ≥0.01 in.;
$NPSpeed_n$ = average nth percentile speed for hours with no precipitation; and
m, b = coefficients for nth percentile TTI.

$NPSpeed_n$ is calculated using the relationship shown by Equation A.7:

$$NPSpeed_n = \frac{FFS}{NPTTI_n} \tag{A.7}$$

Table A.3 lists the coefficients corresponding to the five TTI percentiles with snow.

The variable $SSpeed_n$ can be converted back into a TTI for snow by using Equation A.8:

$$STTI_n = \frac{FFS}{SSpeed_n} \tag{A.8}$$

Final Reliability Model Incorporating Rain and Snow

On the basis of the number of days of each type of precipitation, a weighted average TTI can be calculated, as shown by Equation A.9:

$$TTI_n = \frac{NPdays * NPTTI_n + Raindays * RTTI_n + Snowdays * STTI_n}{365} \tag{A.9}$$

Table A.3. Coefficients Corresponding to nth Percentile TTI: Snow

TTI Percentile	m	b
10	0.178	15.55
50	0.345	3.27
80	0.233	5.24
95	0.286	1.67
99	0.341	–0.55

where

TTI_n = nth percentile TTI for a 1-h time-slice over 1 year;
NPdays = number of days (for a 1-h time-slice) with no precipitation;
$NPTTI_n$ = nth percentile TTI for days with no precipitation;
Raindays = number of days (for a 1-h time-slice) with rain ≥0.05 in.;
$RTTI_n$ = nth percentile TTI for days with rain ≥0.05 in.;
Snowdays = number of days (for a 1-h time-slice) with snow ≥0.01 in.; and
$STTI_n$ = nth percentile TTI for days with snow ≥0.01 in.

Analyze Existing Data to Improve Applicability of Reliability Models for Time Periods with *d/c* < 0.8

The objective of this effort was to improve the applicability of reliability models for periods with *d/c* ratios less than 0.8. The Project L03 reliability model that was of most use to Project L07 for evaluating design treatments was the peak hour model. Originally, the research team anticipated being able to use the L03 peak hour model without adjustment to calculate the TTI distribution for each hour of the day. However, applying the peak hour model to an hour with a low *d/c* yielded unrealistic results. These results were likely because the subset of data used to create the peak hour model in Project L03 included only peak hour (i.e., congested) data, and freeway sections with peak hours of *d/c* < 0.8 are relatively rare. Because of this, the existing models show an effect on nonrecurrent congestion only during peak time periods when there is also substantial recurrent congestion. This limitation meant that the available reliability models were very applicable to peak periods on freeways in major metropolitan areas, but had limited applicability to off-peak periods on freeways in major metropolitan areas, peak and off-peak periods in medium and smaller metropolitan areas, and peak and off-peak conditions on rural freeways.

The research team analyzed the data that were already available from Project L03 for time periods with *d/c* < 0.8 to better quantify the contributions of incidents during those periods to travel time reliability. For every hour of the day (not just the peak hours), the research team calculated values for the model input variables (*d/c*, LHL) and compared the observed cumulative TTI curves with the predicted cumulative TTI curves (i.e., predicted with the L03 reliability models). In some cases, the observed and predicted curves were very similar; in other cases, they were markedly different.

The Project L03 research team had developed the reliability models based on roadway sections, but the Project L07 research team conducted the analysis at the link level. (A *link* is defined as a continuous portion of freeway between an on-ramp and the next off-ramp; a *section* is a group of several consecutive links.) Because the input format for the Analysis Tool (one of the major deliverables of the L07 project) is at a link level, a model to predict cumulative TTI curves for a single link was deemed more applicable. Figure A.3 shows the distribution of link lengths for the Minneapolis–St. Paul data.

Based on the development of the Project L03 models, the time period for predicting a TTI distribution should be 1 year. So, the models predict operations for a time-slice (e.g., 8:00 to 9:00 a.m.) over an entire year. To develop the $d/c < 0.8$ model, the raw data were transformed into this form. Each row of the database represented a single hour for the entire year at a given link. A single combination of hour–year–link (HYL) made up one data point in the database. Most of the raw data came in 5-min, by lane volumes and speeds, so a procedure was developed to filter and sum the data appropriately. For each HYL, the following values were determined on the basis of the observed data:

- 99th percentile TTI
- 95th percentile TTI
- 80th percentile TTI
- 50th percentile TTI
- 10th percentile TTI

These results were first used to verify that using the Project L03 models for periods with $d/c < 0.8$ did not produce sufficiently accurate results. The research team compared the observed values for each percentile with the predicted TTI values by using the L03 reliability models. The charts shown in Figure A.4 display an observed TTI distribution and a L03-predicted TTI distribution for several HYL combinations. In some cases, the observed and predicted curves are nearly identical; in others, they are significantly different.

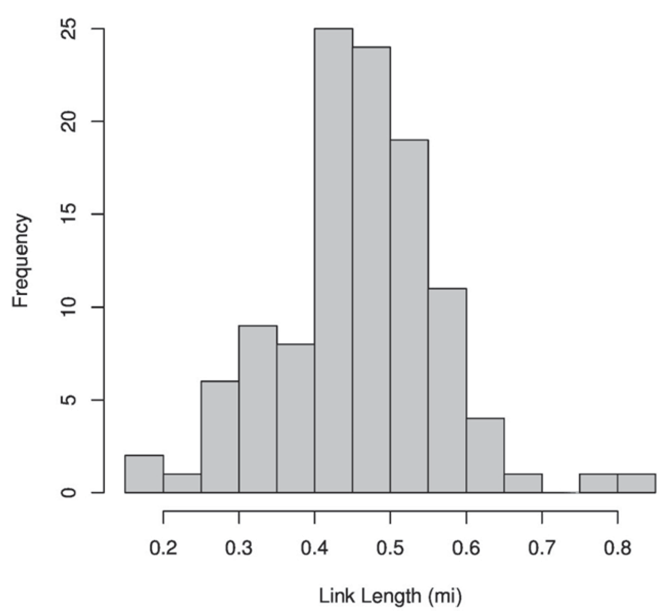


Figure A.3. Histogram of link lengths for Minneapolis–St. Paul data.

Figure A.5 shows the relative error (|observed − [predicted/observed]|) of the 95th percentile prediction model for the 2006 Minneapolis–St. Paul data. Many of the points are above a relative error of 0.3, meaning that the prediction is off by over 30%.

Figure A.5 also shows the largest TTI prediction error in the d/c range between 0.4 and 0.8. Similar graphs were created for the other four percentiles. Together, these graphs show that TTI distributions for HYL combinations with very low d/c values tend to vary widely. This variation is likely due to rare catastrophic incidents, because at such low d/c levels, it is unlikely that a bottleneck could be created by something other than an incident that blocks several lanes. The research team therefore concluded that the best range to model TTI percentiles is between 0.4 and 0.8 d/c. This range of the observed data was extracted from the database and used by the statisticians on the research team to generate models appropriate to this range.

Independent Variables

To create a model to predict the five percentile values when d/c is less than 0.8, the independent variables for each HYL were identified and calculated. The first three independent variables were identified in Project L03: d/c, LHL, and $R_{0.05''}$. The fourth independent variable ($S_{0.01''}$) was developed and included in the model as part of Project L07. $S_{0.01''}$ is defined as the number of hours per year during a particular time-slice when snowfall exceeds 0.01 in.

Demand-to-Capacity Ratio

The capacity for each link was obtained from a Project L03 database. Demand was not readily available and had to be determined from observed volumes and speeds and by calculating median densities and 15th percentile speeds.

Lane Hours Lost

LHL represents the sum of the time when a lane (or shoulder) is blocked by a crash-involved or disabled vehicle or by a work zone. The raw data from Minneapolis–St. Paul included records from a traffic management center that kept records of lane-blocking and shoulder-blocking events. The type of event for each record was determined and assigned the appropriate duration and lane-blocking space to each hour.

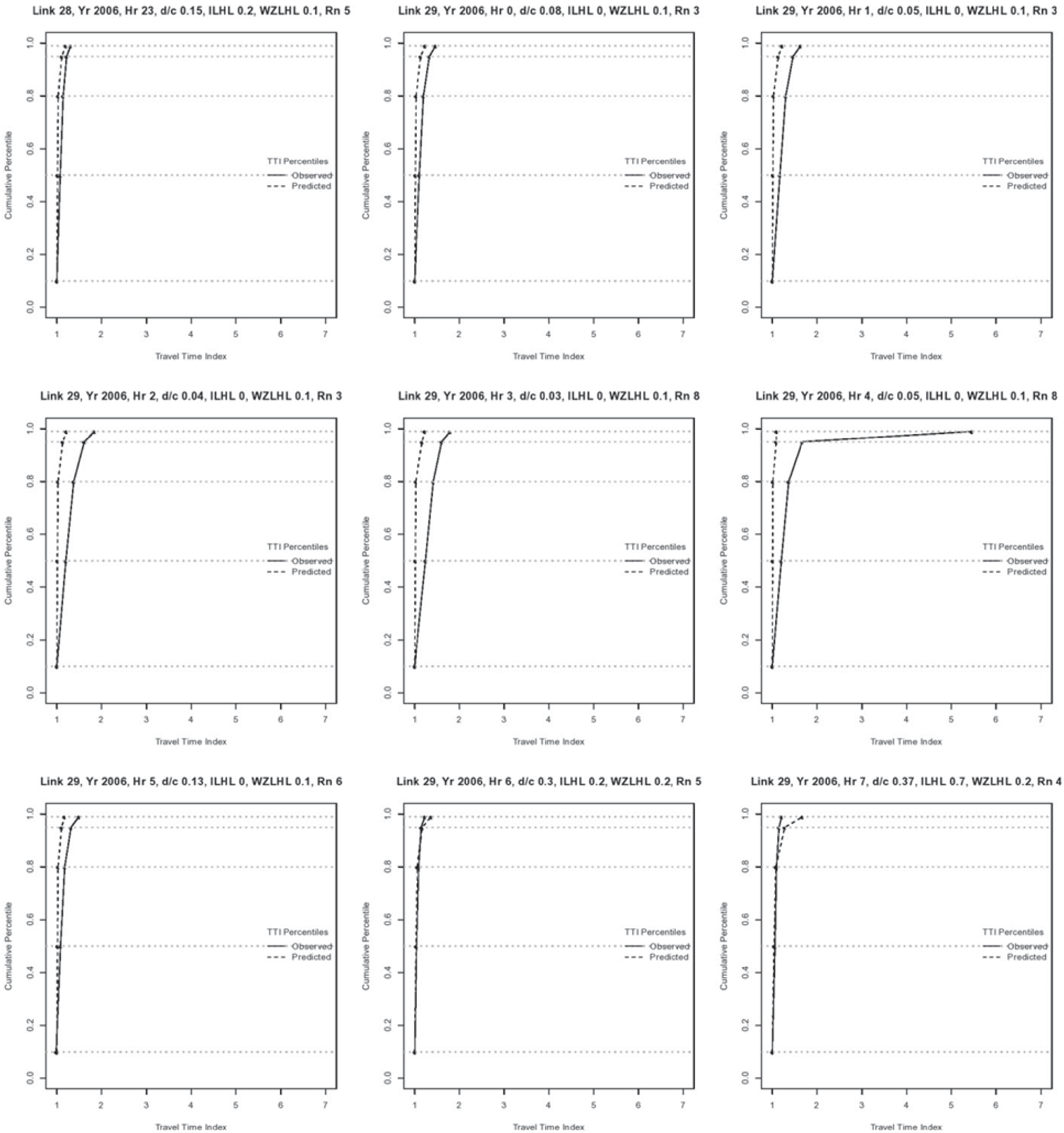


Figure A.4. Excerpt of figures comparing observed with L03-predicted TTI distributions.

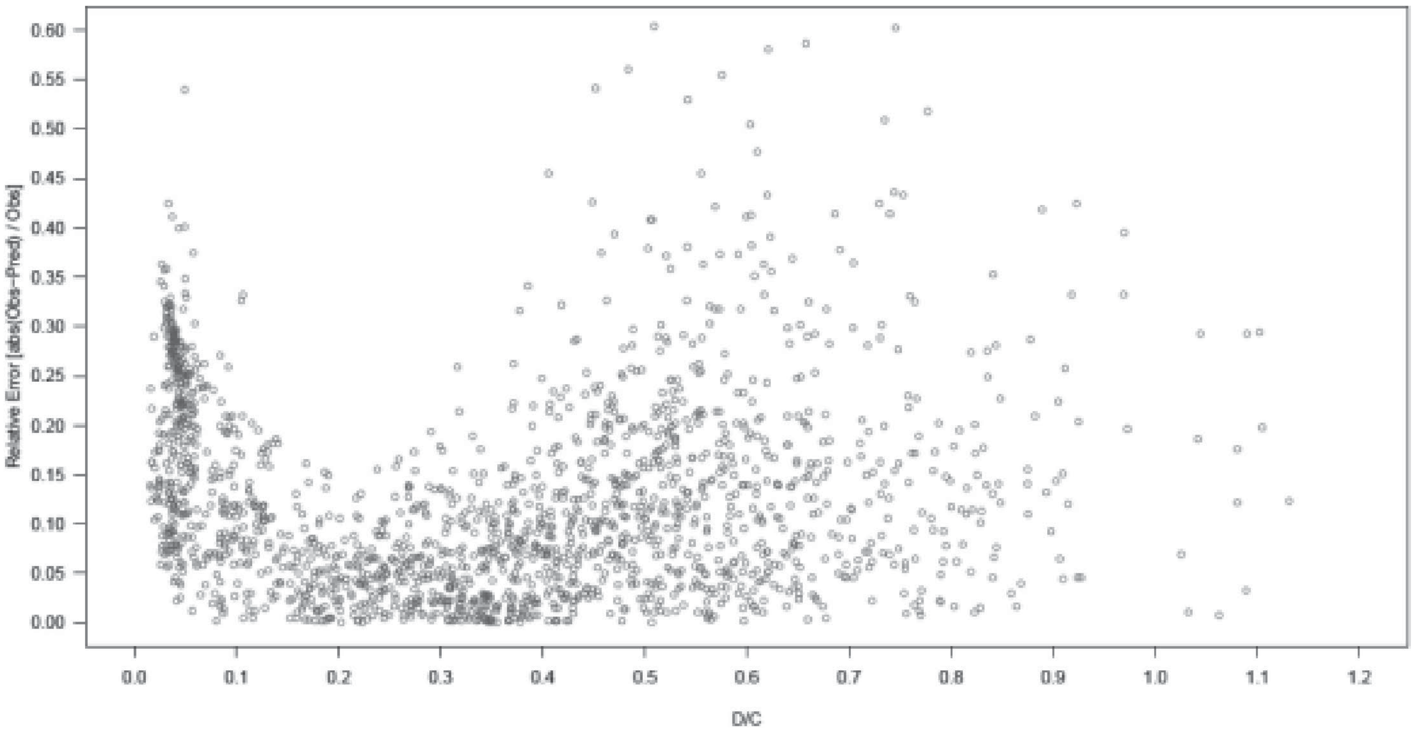


Figure A.5. Relative error of L03 95th percentile TTI models.

Total work-zone lane hours lost (WZLHL) for a given hour was determined by summing all incidents of the following categories from the incident database: scheduled construction and unscheduled construction. However, before these LHLs could be assigned to links, an assumption had to be made about the length of a typical work zone. If work zones typically block only one link, they were distributed just as incidents were above, by link length alone. However, if work zones typically block all links within a section, the total WZLHL value was multiplied by the number of links in the section and then distributed by link length. The research team randomly selected 12 construction events and manually matched each event to the right segment of freeway. The beginning and ending points of each link were also identified on the map to determine how many links were blocked by the construction event. The results of this analysis are shown in Table A.4.

As shown in Table A.4, the percentage of a roadway section that was blocked by construction events varied from 14% to 100%. Ideally, one would analyze each work-zone event individually, and the WZLHL would be assigned only to those links actually affected by the work zone. However, because of the time-consuming nature of such an effort, a simplifying assumption was made. The total WZLHL for a section was multiplied by the number of links in that section and then multiplied by 50%. The total WZLHL was then distributed to each link based on the link length as a proportion of total section length.

Figure A.6 shows the resulting total LHL values (including WZLHL and LHL due to incidents) per hour with all links combined and averaged by mile. Each bar represents an hour in 2006 (Hour 0 is midnight, and Hour 23 is 11:00 p.m.). The bars are shaded by LHL cause to show the types of events that contributed most to the total LHL value.

Table A.4. Number of Links Blocked by Construction Events

Construction Event	No. of Links Blocked by Construction Event	Total No. of Links in Section	Section Blocked (%)
1	2	8	25
2	7	7	100
3	6	11	55
4	5	11	45
5	5	11	45
6	5	11	45
7	5	11	45
8	2	7	29
9	3	7	43
10	3	7	43
11	1	7	14
12	1	7	14

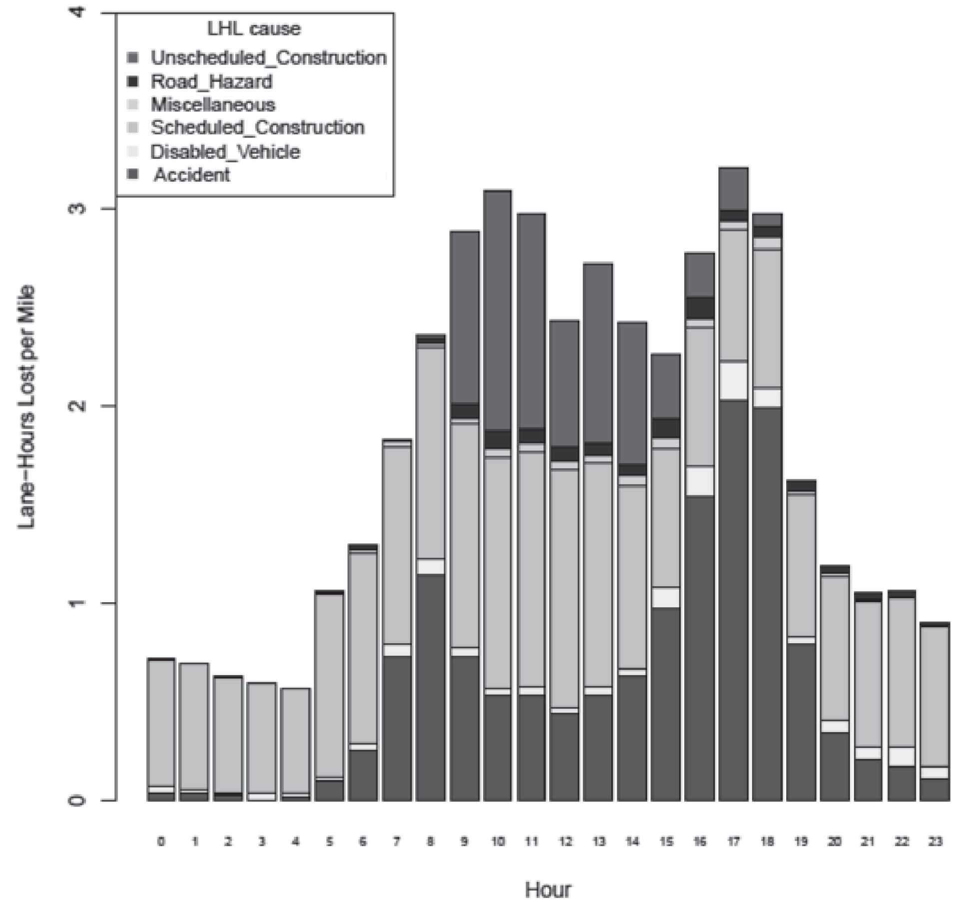


Figure A.6. Average LHL per mile by hour.

$R_{0.05''}$ and $S_{0.01''}$

$R_{0.05''}$ is the number of times (during a given hour over the course of 1 year) that rainfall exceeds 0.05 in. $S_{0.01''}$ is the number of times snowfall exceeds 0.01 in. within the same time parameters. The values of $R_{0.05''}$ and $S_{0.01''}$ were determined using data from the National Weather Service. Four weather stations with complete data for the years of interest were identified in Minneapolis–St. Paul. Using Microsoft Streets and Trips software, the location of each weather station and each link was plotted on a map. The weather station nearest to each link was recorded, and data from that weather station were used to calculate the $R_{0.05''}$ and $S_{0.01''}$ values to be used for that link. $R_{0.05''}$ values in the Minneapolis–St. Paul area ranged from two to 10, and $S_{0.01''}$ values ranged from zero to six.

Final Models

The final database included 1,810 records. Each record represented a single HYL and included values for *d/c,* LHL due to incidents, WZLHL, $R_{0.05''}$, and $S_{0.01''}$ for that HYL. The observed 10th, 50th, 80th, 95th, and 99th percentile TTIs for each HYL were calculated and displayed in the final database. This database was used to create the following models for predicting TTI values based on the four input variables (*d/c,* LHL, $R_{0.05''}$, and $S_{0.01''}$). These models retain the form used in L03 (exponential). The general form is as shown in Equation A.10:

$$\mathrm{TTI}_i = e^{\left(a_i \times \frac{d}{c}\right) + (b_i \times \mathrm{LHL}) + (c_i \times R_{0.05''}) + (d_i \times S_{0.01''})} \qquad \text{(A.10)}$$

where TTI_i is the cumulative TTI at percentile i; a_i, b_i, c_i, and d_i are coefficients at percentile i; and other variables are as defined previously.

The coefficients *a, b, c,* and *d* are calculated for a given percentile by using Equation A.11:

$$\text{coefficient}_i = (w \times i) + \left(x \times y^{[z(i-1)]}\right) \qquad \text{(A.11)}$$

where i is a given percentile value (between 0 and 1), and *w, x, y,* and *z* are constants (see Table A.5).

Table A.5. Constants for Equation A.11

	w	x	y	z
a	0.14	0.504	96	9
b	0.0099	0.0481	96	9
c	0.00149	0.0197	68	6
d	0.00367	0.0248	36	7

Related SHRP 2 Research

Establishing Monitoring Programs for Travel Time Reliability (L02)

Analytical Procedures for Determining the Impacts of Reliability Mitigation Strategies (L03)

Incorporating Reliability Performance Measures into the Transportation Planning and Programming Processes (L05)

Incorporating Travel Time Reliability into the *Highway Capacity Manual* (L08)

Evaluating Alternative Operations Strategies to Improve Travel Time Reliability (L11)

Development of Tools for Assessing Wider Economic Benefits of Transportation (C11)