

SHRP 2 Reliability Project L07

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

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FINAL REPORT

**Prepared for
The Strategic Highway Research Program 2
Transportation Research Board
of
The National Academies**

**Ingrid B. Potts, Douglas W. Harwood, Jessica M. Hutton,
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August 2013

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Abstract

SHRP 2 Project L07 has focused specifically on design treatments that can be used to reduce delays due to nonrecurrent congestion and improve travel-time reliability. The objectives of this research are to (1) identify the full range of possible design treatments used by transportation agencies to improve travel-time reliability and reduce delays due to 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.

Three separate analyses of the design treatments were conducted in order to achieve the research objectives: operational, safety, and benefit-cost. The traffic operational analysis methodology developed in this research built from work completed in SHRP 2 Project L03. For the safety analysis of nonrecurrent congestion treatments, this research explored the relationship between congestion and safety—specifically the relationship between Level of Service and crash frequency—and developed a mathematical model to quantify the increase in crash frequency at all severity levels as LOS worsens.

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.

Executive Summary

The Reliability area of the SHRP 2 program has focused on the need to improve travel-time reliability on freeways and major arterials. SHRP 2 Project L07 has focused specifically on design treatments that can be used to improve travel-time reliability. The objectives of this research are to (1) identify the full range of possible roadway design features used by transportation agencies to improve travel-time reliability and reduce delays due to 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 focuses 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 (ITS) treatments. That is, daily demand peaks that cause rush 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 through traffic management centers that 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, there is a gap in the literature regarding the use of geometric design treatments to help reduce nonrecurrent congestion, which the research in Project L07 helps fill.

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, and this was frequently done in an ad hoc fashion. When major incidents occurred, agencies used whatever tools were at their disposal to minimize the disruption to traffic. While 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 the lanes, adding temporary capacity to alleviate congestion (for example, allowing shoulder driving), providing opportunities for vehicles to escape a queue and find a new route (such as 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 (such as anti-icing systems).

Three separate analyses of the design treatments were conducted in order to achieve the research objectives: operational, safety, and benefit-cost. The traffic operational analysis methodology developed in this research built from work completed in SHRP 2 Project L03, which preceded this research effort. That research effort developed models for predicting a travel

time index (TTI) at five percentiles (10th, 50th, 80th, 95th, 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 our purposes, a 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 a 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 L03 Project 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 hours of the day. This research adapted the Project L03 models for use during one-hour time-slices, so that the TTI distribution could be predicted for each hour of the day. In addition, the research team improved upon the 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 ($S_{0.01}$) 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/or hours of the day with lower demand-to-capacity ratios (i.e., less than 0.8).

The TTI models as modified for this 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 a TTI curve for 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 design treatments have an effect on the “lane hours lost” variable by minimizing the number of incidents that occur, reducing the time incidents and work zones block lanes, 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 have an impact on the demand-to-capacity 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 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 LOS C, but then begins increasing through LOS D, E, and F. This relationship indicates that

if improvements can be made to level of service (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 will 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 using the safety-congestion relationship.

The third treatment analysis was a benefit-cost evaluation for the various design treatments that were evaluated. 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 based on the shift in TTI cumulative curves from untreated to treated conditions. In this project, reliability is 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. We 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. Design treatments that are commonly used to address recurrent congestion can also 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 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 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. The Design Guide serves as a catalog of 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 based on user-inputted information 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

1.1 Background

The Reliability area of the SHRP 2 program has focused on the need to improve travel-time reliability on freeways and major arterials. The objectives of this research are to (1) identify the full range of possible roadway design features used by transportation agencies to improve travel-time reliability and reduce delays due to 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 focuses on geometric design treatments that can be used to reduce delays due to nonrecurrent congestion.

1.1.1 Recurrent Congestion vs. 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 is most frequently associated with commute 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 design or redesign of highways, bridges, and intersections on which it has occurred or is expected to occur.

Nonrecurrent Congestion

Nonrecurrent congestion is congestion arising from random events generally unpredictable to the facility user, varying in degree from day to day and from one incident to the next, creating unreliable travel times that frustrate motorists. Sources of nonrecurrent congestion include:

- *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/or 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 results 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. They are events that cause the traffic flow in the vicinity of the event to be radically different 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 (ITS) 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.

While *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.

1.1.2 Reliability—Definition 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 a motorists’ perception of a trip because information allows a motorist 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. A definition of reliability and key terms related to reliability are presented below.

Definition of Reliability

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

The research team for SHRP 2 Project L03, *Analytic Procedures for Determining the Impacts of Reliability Mitigation Strategies (I)*, developed a working definition for reliability, and this definition has been adopted in Project L07:

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 has been defined in Projects L03 and L07 as *1 year* for convenience and practicality. A period of 1 year also ensures a substantial enough dataset 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 multi-hour portion of a 24-hour day, considered over an entire year (excluding weekends and holidays). For example, “the hour from 6:00 to 7:00 a.m. for every non-holiday 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.”

Multi-hour time-slices defined and evaluated by Project L03 include:

- *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 hours aggregated.

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

Travel Time Index (TTI)

While 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 length is desirable. In reliability research, the Travel Time Index (TTI) has emerged as such a measure. The TTI is defined as follows:

Travel Time Index (TTI): *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-mile freeway segment is 60 mph (meaning a vehicle could traverse the segment in 2.0 minutes), and a vehicle traverses the segment in 2.4 minutes, then the TTI for that vehicle is the ratio of 2.4 to 2.0 minutes, or $TTI = 1.2$. (**NOTE:** 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 could be considered as a fundamental measure 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 (for example, seconds per mile).

However, as a unitless measure, TTI is a preferable standard because it can compare across different facilities regardless of the speed they are designed for. For example, an average travel speed of 55 mph (travel rate of 65 sec/mi) would be quite acceptable on a facility with a free-flow speed of 55 mph (65 sec/mi), but would be less acceptable on a facility with a free-flow speed of 70 mph (51 sec/mi). In each case, the analyst would need to be presented with 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.

Since 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) is used as a surrogate for free-flow speed, and any TTI values less than 1.0 are set equal to 1.0.

1.1.4 The 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.

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

1.1.5 The Fundamental Diagram of Reliability

As stated previously, 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).

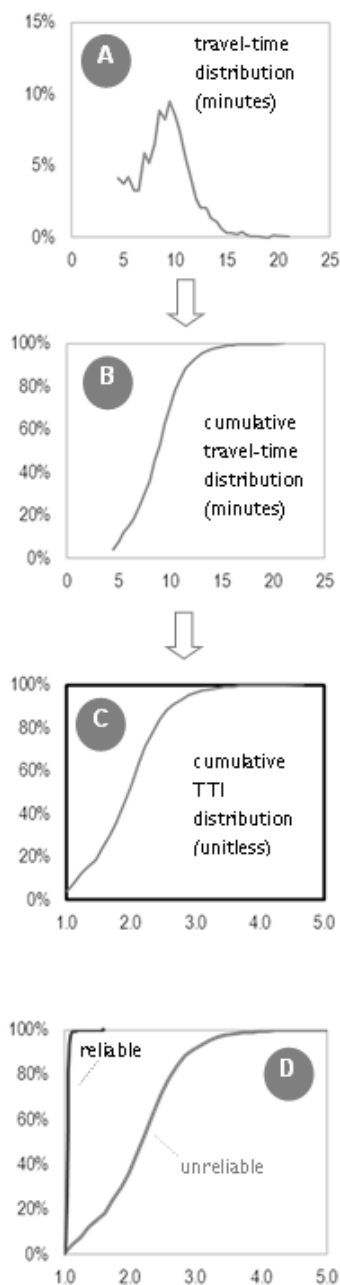


Figure 1-1. Cumulative TTI Curve

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 travel time index 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 indicates that there is little difference in the travel times between the shortest trips and the longest trips, and that the travel time index for even longer trips is close to 1.0. Graph D illustrates this principle.

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

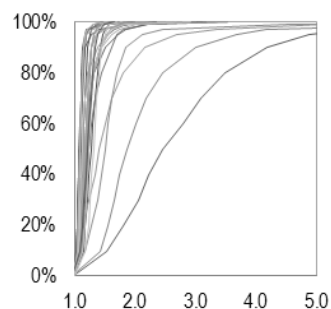


Figure 1–2. Sample 24-Hour Cumulative TTI Curves

Mean-Based Measures

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

Mean TTI

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,” while 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 author (*I*) 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

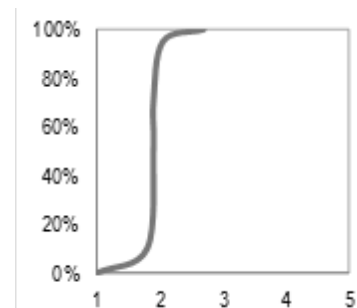


Figure 1-3. Reliable but Undesirable: A Theoretical Construct

capacity, delay values are very volatile, and so the 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, while 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} , the result is the total delay experienced by all vehicles:

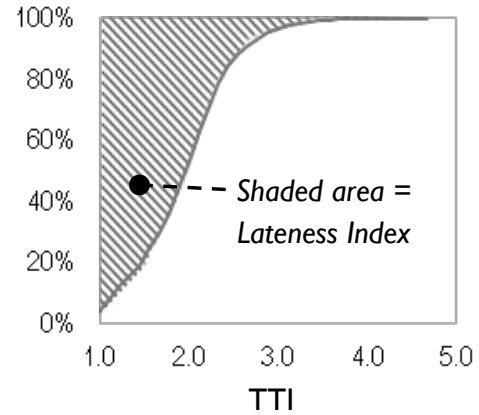


Figure 1–4. Lateness Index

$$\text{Total Delay} = LI \times V \times TT_{FF} \quad (1)$$

1.1.7 Variance-Based Measures

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

Variance/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, since reliability is primarily concerned with variability.

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

$$\sigma = \frac{1}{n} \int_{i=0}^n (TTI_i - TTI_{mean})^2 di \quad (2)$$

Semi-Variance

Although it is fairly common statistical practice to calculate the variance about the mean (as above), 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 *semi-variance* (σ_r) has been used. Statistically this can be described as the second moment of the travel-time distribution about the minimum:

$$\sigma_r = \frac{1}{n} \int_{i=0}^n (TTI_i - r)^2 di = \int_{i=0}^{100\%} (TTI_i - 1)^2 di \quad (3)$$

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 $(TTI - TTI_{mean})^2$ and semi-variance $(TTI - 1)^2$. The variance curve is constructed from the cumulative TTI curve by calculating $(TTI - c)^2$, for each percentile p , the difference between TTI_p and the vertical line $y = TTI_{mean}$, and then squaring that difference. The semi-variance 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 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 previously 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.

NOTE: *The Lateness Index and the Semi-Variance provide roughly the same information about the cumulative TTI curve—the former being a summation of $(TTI - 1)$ and the latter being a summation of $(TTI - 1)^2$. The Semi-variance places disproportionate emphasis on larger deviations, and therefore may better gauge reliability.*

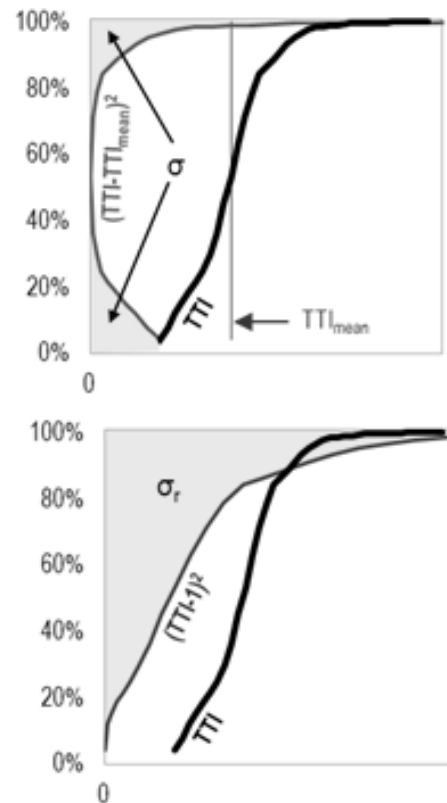


Figure 1–5. Variance and Semi-Variance Buildups

Single-Point/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.

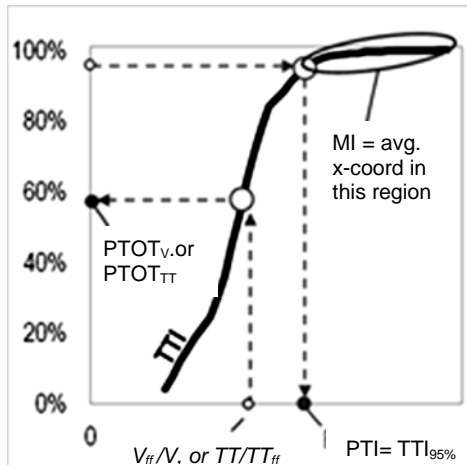


Figure 1-6. Single-Point/Regime Indices

Planning Time Index (PTI)

The 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 percent of the time.

Misery Index (MI)

The MI represents the average of the highest 5 percent 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. The MI

may be especially useful in characterizing rural reliability, wherein 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.

Percent Trips on Time (PTOT)

This measure 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. PTOT represents the percent of trips completed within a certain speed or time range—for example, the percent of trips that arrived on time with a speed of 45 mph or greater or the percent 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.

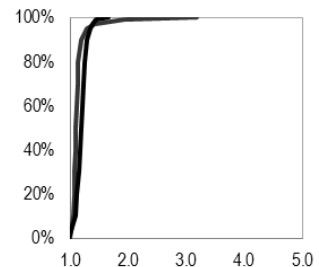


Figure 1-7. Identical PTIs

Curvature Indices

Several reliability measures are built on ratios that describe aspects of the curvature of the cumulative curve. Figure 1-8 (top graph) illustrates these measures in relation to the cumulative curve.

Buffer Index (BI)

The 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, the BI hinges on the 95th-percentile TTI, but uses a ratio involving either the mean or the median:

$$BI_{\text{mean}} = (TTI_{95\%} - TTI_{\text{mean}}) / (TTI_{\text{mean}}) \quad (4)$$

$$BI_{50\%} = (TTI_{95\%} - TTI_{50\%}) / (TTI_{50\%}) \quad (5)$$

Recent research (1) has raised doubts about the use of the Buffer Index as a primary reliability metric for tracking trends in reliability, due to its erratic and unstable nature. Treatments that tend to uniformly decrease travel times (rather than affecting only the extremes) can result in counterintuitive Buffer Indices (falsely indicating reliability degradations when conditions are actually improving). However, the Buffer Index remains useful as a secondary metric.

Skew Statistic (SS)

The 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:

$$SS = (TTI_{90\%} - TTI_{50\%}) / (TTI_{50\%} - TTI_{10\%}) \quad (6)$$

Measures such as the BI and SS, while providing information about the shape of the travel-time distribution, do not provide sufficient information about the *desirability* of the distribution. The lower two graphs in Figure 1–8 illustrate that very different distributions can have the identical Buffer Indices 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, wherein an index (level of service, or LOS) 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 the Lateness Index and Semi-Variance best exhibit this behavior. Curvature indices (BI, SS) do not do so reliably. Point measures cannot always tell the

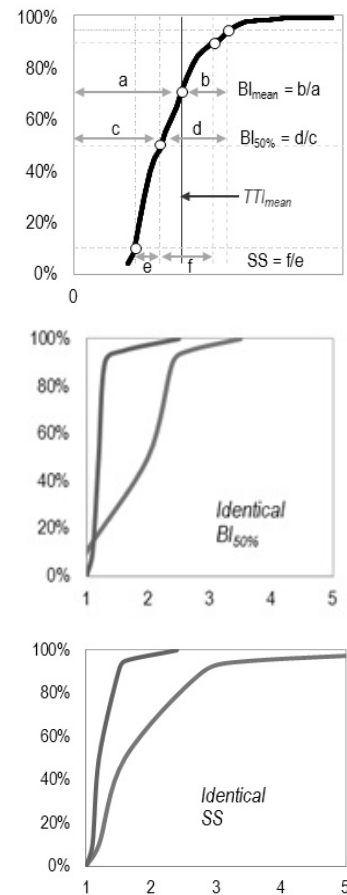


Figure 1–8. Curvature Indices

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 vs. arterial).

1.1.8 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 (ramp-metering implementation). The shaded area is equal to the differences in the Lateness Index, 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.

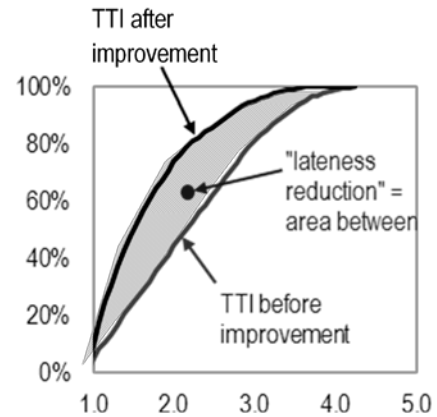


Figure 1–9. Delay Reduction

The delay reduction above can further be translated into economic terms using the monetary value of time (VOT). Research has also 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 using a monetary value of reliability (VOR).

1.1.9 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/environment attributes (I):

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

These predictive formulas are the foundation for most of the operational analysis work of Project L07, as will be explained in Chapter 4.

1.2 Organization of this Report

The remainder of this report is organized as follows:

Chapter 2 Research Approach—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 provides a brief summary of the three research products in addition to this final report: the Design Guide, Analysis Tool, and Dissemination Plan.

Chapter 3 Data Collection and Documentation of Current Design Practice—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 presents a list of all the nonrecurrent congestion design treatments considered in this research.

Chapter 4 Traffic Operational Assessment—Chapter 4 explains in mathematical terms how the predicted travel time index 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) is presented in detail.

Chapter 5 Safety Assessment of Design Treatments—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 HCM procedures, and other roadway and roadside design features that may impact 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 Life-Cycle Benefit-Cost Analysis—Chapter 6 describe 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.

Chapter 7 Test of Reasonableness of Tool Outputs—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.

Chapter 8 Conclusions and Recommendations—Major findings from all phases of the research are summarized in Chapter 8. These conclusions came from not only 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. The chapter concludes with recommendations for how the results of this research might be implemented by highway agencies.

References—The references cited in Chapters 1 through 8 are listed here.

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.

2.1 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 due to 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, while 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 in order to achieve the research objectives. The primary analysis was a traffic operational assessment, which 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 a treatment is implemented. 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. In addition, a secondary analysis of the safety implications of using the design treatments was conducted. While 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 (LOS) 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.

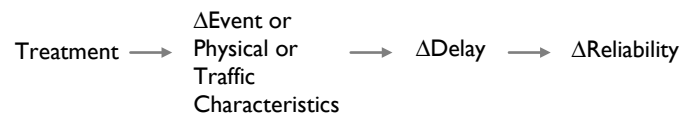
The traffic operational analysis methodology developed in this research was intended to build from work completed in SHRP 2 Project L03. However, the products of Project L03 did not exactly meet the needs of the Project L07 analysis. The next section describes the evolution of the research approach for 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.

2.2 Evolution of Research Approach for Traffic Operational Analysis

Our 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:

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



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 they 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. Foremost among these conditions is that the Project L03 models are most applicable to urban freeways in major metropolitan areas, whereas the scope of Project L07 includes rural and small/medium urban areas as well. In addition, the Project L03 models are most applicable to peak periods, whereas Project L07 focuses 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 includes time-slices (i.e., portions of the day) in which the demand-to-capacity (d/c) ratio is 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/medium 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 research team and SHRP 2 hoped that reliability models could be developed for the full range of d/c; that is, for congested and uncongested periods. Furthermore, the Project L03 models included a variable, $R_{0.05}$, to account for rainfall 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 spreadsheet-based analysis tool that were developed earlier in the project. Specifically, the extension of the project focused on:

- 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 multi-hour 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

2.3 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.

2.4 Design Guide

The Design Guide serves as a catalog of 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 list of design treatments that were considered in this research are presented with a decision tree that assists the user in narrowing down the full list to a shorter list of design treatments 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 objective
- 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 then 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 cover-to-cover, as its main function is to serve as a catalog of nonrecurrent congestion treatments, which 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 which may be applicable to a specific roadway of interest to investigate further using the Analysis Tool discussed below.

2.5 Analysis Tool

As noted above, the 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 (geometrics, volumes, crash totals, etc.), 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 is a VBA interface (see Figure 2–1) 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 Project L07 Final 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. Therefore, this tool and its successors will become more sophisticated in the future. Nevertheless, the tool is a comprehensive approach to applying the principles developed in Project L07.

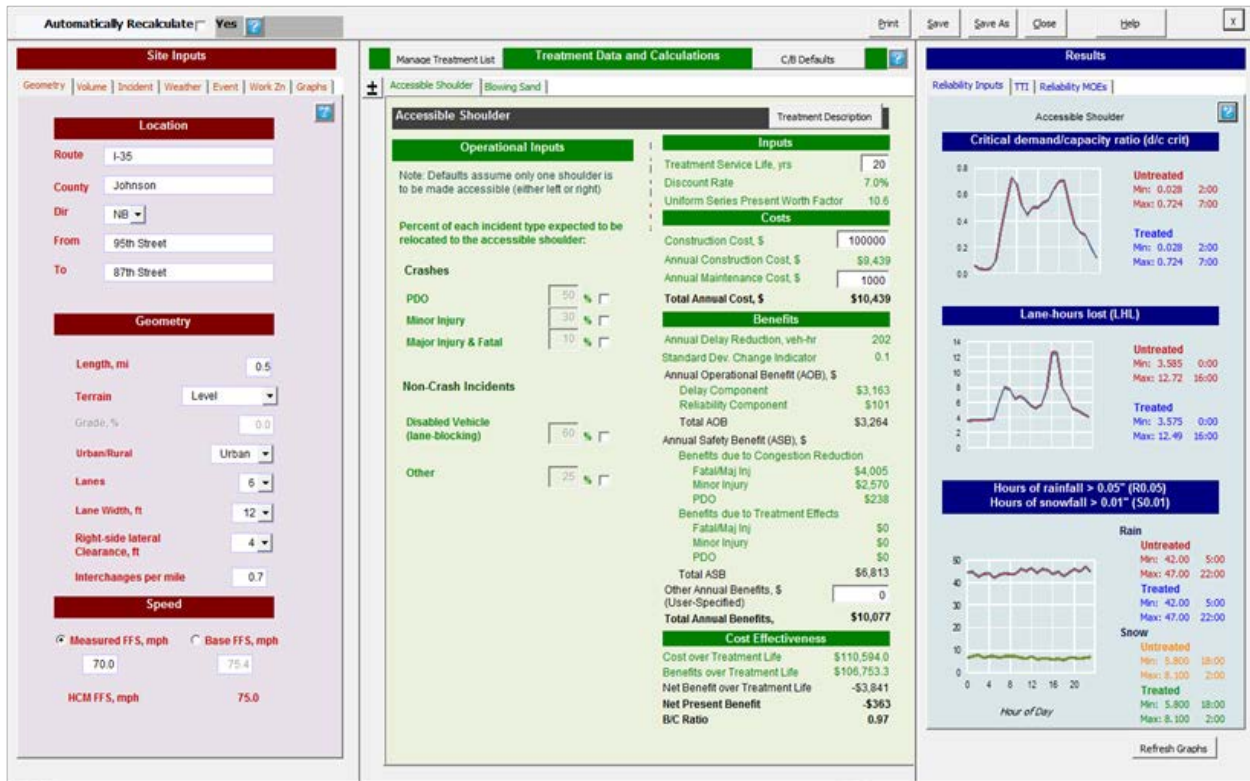


Figure 2–1. Project L07 Spreadsheet Analysis Tool—User Interface

The tool interface is divided into three parts, as can be seen in Figure 2–1:

- *Site Inputs:* The user enters data regarding location (segment name, length, etc.), geometry (number of lanes, lane widths, grade, etc.), demand (hourly demand, peak-hour factors, and truck percentages for a typical 24-hour day), special event information (hourly volume percent 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/type)
- *Treatment Data and Calculations:* The user enters specific data regarding each selected treatment's effects, including percent of incidents reduced by type, the effects of the treatment on average incident duration, etc. 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 (B/C) 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 (MOEs).

2.6 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 are to:

- Increase awareness of Project L07's research findings, including the benefits and value of the design guidebook and analysis tool within the transportation community.
- Spur the adoption and integration of the design guidebook 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; and
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:

- 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/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 upon 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:

- 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 email 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 list of design treatments for evaluation

This section of the report presents a summary of each of these activities.

3.1 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 (*t*). (A time-slice can be a one-hour or multi-hour 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 (buffer time, planning time, etc.) 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 based on three key variables:

- *Lane-hours lost (LHL)* due to incidents and work zones. This value 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 (dc_{crit})*, 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 ($R_{0.05''}$)* 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 include: peak hour, peak period, mid-day (the 2-hour period from 11:00 a.m. to 1:00 p.m.), and weekday (all 24 hours). Because nonrecurrent congestion can occur at any time of the day, Project L07 needed a relationship that covered each of the 24 hours 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.)

The Project L03 relationships have the following general functional form:

$$TTI_{n\%} = e^{(j_n LHL + k_n dc_{crit} + l_n R_{0.05''})} \quad (7)$$

where: $TTI_{n\%}$ = n th-percentile TTI value
 LHL = lane-hours lost
 dc_{crit} = critical demand-capacity ratio
 $R_{0.05''}$ = hours of rainfall exceeding 0.05 in
 j_n, k_n, l_n = coefficients for n th percentile (see Table 3-1)

Table 3-1. Coefficients Used in Project L03 Reliability Models—Peak Hour (3)

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 .

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—most unreliable—hour of the day) has a 50th-percentile TTI of 2.3, signifying that 50 percent 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.

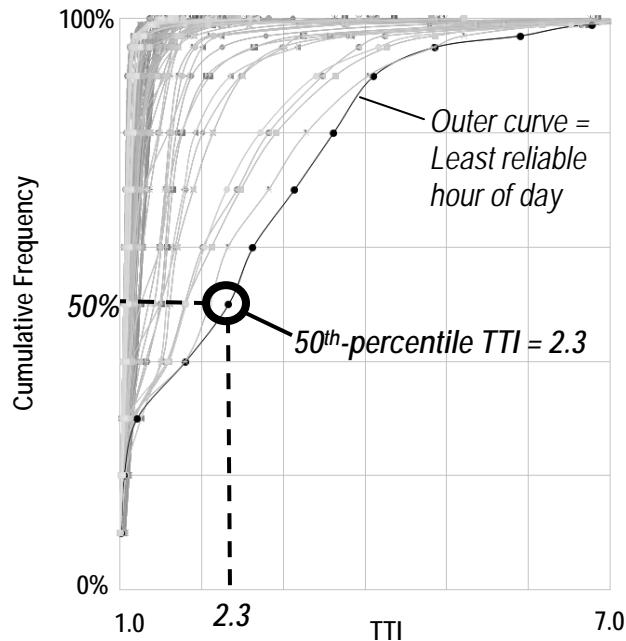


Figure 3–1. Cumulative TTI Distribution Per Hour of Day

3.2 Assembly of Databases

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

- 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.

3.2.1 Traffic Operational Data

Three (3) 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-minute volume data per travel lane and 5-minute 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. While this period might have been interesting (because volumes changed dramatically on many freeway segments), the changed driving conditions were new to many drivers and Mn/DOT made many modifications to specific roadways to increase base capacity; thus, this time period would likely include unusual flow conditions.

3.2.2 Crash Data

Crash data for each directional freeway segment were obtained through HSIS. The crash 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:

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

3.2.3 Weather Data

The research team obtained 10 years (2001 through 2010) of hourly precipitation data across the U.S. from the National Climactic Data Center (NCDC). From the NCDC website, the research team downloaded Local Climatological Data LCD/Quality Controlled databases for all stations within the U.S.

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.



Figure 3–2. U.S. Weather Stations With Data Available
 (©Microsoft Streets and Trips)

3.3 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 (ITS) 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.

3.4 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 information about design treatments. Telephone interviews were then conducted with knowledgeable engineers in the most promising agencies. The highway agencies that were contacted include 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

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 has any suitable projects or sites for evaluation in the research.

3.5 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 Department of Transportation (Mn/DOT)
- *Atlanta*—Georgia Department of Transportation (GDOT)
- *Baltimore/Washington, DC*—Maryland Department of Transportation (MDOT)
- *New York City/Newark*—Port Authority of New York and New Jersey (PANYNJ)

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 we conducted, they were actively involved in using design treatments to address delay caused by nonrecurrent (as well as recurrent) congestion.

Figure 3–3 summarizes the states that were contacted by email or telephone as well as the metropolitan areas where focus groups were conducted.

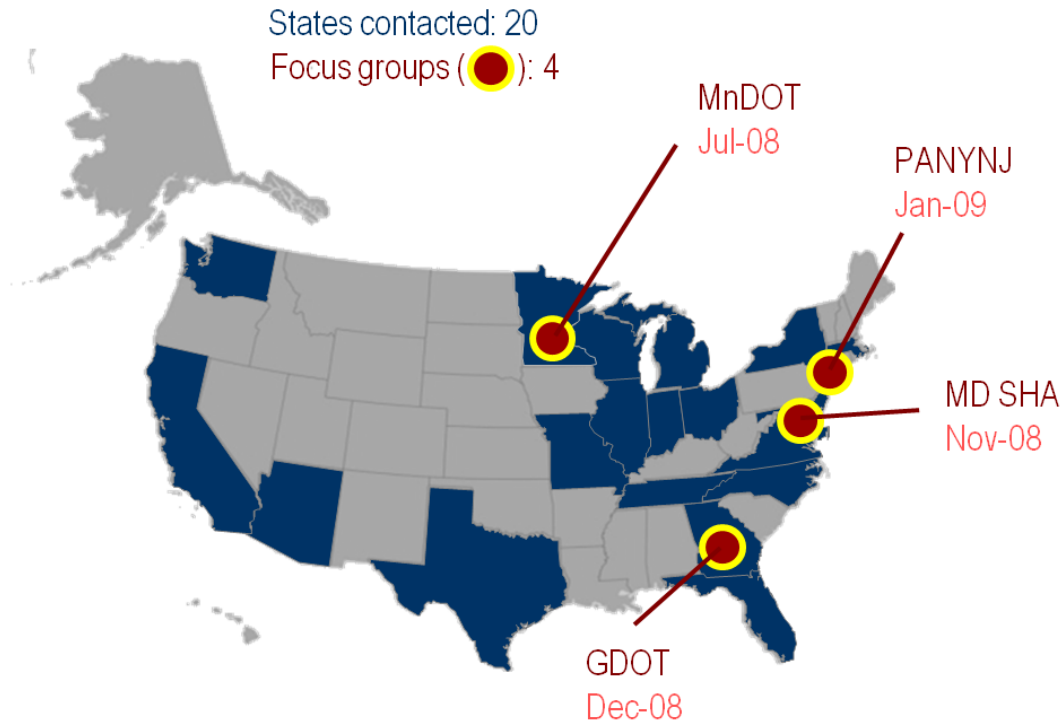


Figure 3–3. States Contacted by Email and/or Telephone That Participated in a Focus Group

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:

- 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 there 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.

3.6 Workshops With Highway Agencies to Discuss Weather Treatments

Since 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, MO, and included weather experts from the following highway agencies/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 lead 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 Mn/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 Mn/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:

- Snow fences
- Road Weather Information Systems (RWIS)
- Anti-icing systems
- Flood warning systems
- Fog detection systems
- Wind warning systems
- Contraflow for hurricane evacuation
- Road closure

3.7 Meetings With Highway Agencies to Obtain Detailed Information About Design Treatments

As part of the focus groups, discussed in Section 3.5, the research team documented highway agency experience with existing treatments and gathered basic information about their design and application. However, in order 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 were:

- *Minnesota Department of Transportation (Mn/DOT)*—The research team met with geometric designers, traffic engineers, and maintenance personnel; the meeting included representatives from Mn/DOT’s incident response program (FIRST) and the Minnesota highway patrol.

- *Illinois Department of Transportation (IDOT)*—The research team met with traffic engineers responsible for traffic operations and incident management in the Chicago metropolitan area.
- *Wisconsin Department of Transportation (WisDOT)*—The research team met with traffic engineers responsible for traffic operations and incident management in the Milwaukee metropolitan area.

The type of crash investigation sites varied greatly between the three agencies. For example, most of the sites that Mn/DOT has constructed in recent years would more appropriately be considered *emergency pull-offs*, because they have been implemented where shoulders are no longer available due to shoulder being converted to travel lanes. The emergency pull-offs serve several purposes, including crash investigation and enforcement, but they were primarily constructed to accommodate broken-down vehicles and other emergencies that would otherwise be accommodated by a shoulder. IDOT’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 we could better estimate the traffic operational effectiveness of crash investigation sites. Key information obtained during the highway agency visits included:

- Typical number of lanes blocked during an incident
- Policy about moving crashes to an emergency pull-off/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/design of an emergency pull-off/crash investigation site
- Typical signing at an emergency pull-off/crash investigation site
- Cost issues with constructing an emergency pull-off/crash investigation site

The research team used this information to develop input variables for the models 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:

- Alternating shoulders
- Bus pull-offs
- Bus shoulders
- Designated bus lanes
- Dynamic shoulders
- Emergency pull-offs
- Emergency traffic operations plan
- Extra-height median barriers

- Flood control systems
- Geometric improvements to alternate routes
- Glare screens
- Median crossovers
- Moveable barriers
- Ramp closure
- Ramp metering
- Reversible lanes
- Traffic signal improvements
- Ramp metering
- Variable speed limits

3.8 List of Design Treatments

Based on 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:

- 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., 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 addresses nonrecurrent congestion and that it supports 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, while 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 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 that highway agencies must understand in order to use this treatment effectively.

Table 3-2. Candidate Design Treatments Considered in the Research

Nonrecurrent congestion design treatments	Secondary treatments
<p>Medians Median crossovers Moveable traffic barriers Gated Median Barrier Extra-height median barriers Mountable/traversable medians</p> <p>Shoulders Accessible shoulder Drivable shoulder Alternating shoulder Portable incident screens Vehicle turnouts Bus turnouts</p> <p>Crash Investigation Sites Crash investigation sites</p> <p>Right-of-way Edge Emergency access between interchanges</p> <p>Arterials and Ramps Ramp widening Ramp closure Ramp terminal traffic control Ramp turn restrictions</p> <p>Detours Improvements to detour routes</p> <p>Truck Incident Design Considerations Runaway truck ramp</p> <p>Construction Reduce construction duration Improved work site access/circulation</p> <p>Animal-Vehicle Collision Design Considerations Wildlife fencing, overpasses, and underpasses, etc.</p> <p>Weather Snow fences Blowing sand treatment Anti-icing systems</p>	<p>Lane Types and Uses Contra-flow lanes for emergency evacuation Contra-flow lanes for work zones HOV lanes/HOT lanes Dual facilities Reversible lanes Work zone express lanes</p> <p>Traffic Signals and Traffic Control Traffic signal preemption Queue jump and bypass lanes Traffic signal improvements Signal timing systems Ramp metering/flow signals Temporary traffic signals Variable speed limits/speed limit reduction</p> <p>Technology Electronic toll collection Over-height vehicle detection and warning systems</p> <p>Emergency Response Notification Reference location signs Roadside call boxes</p> <p>Weather Fog detection Road Weather Information System (RWIS) Flood warning system Wind warning system</p>

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 is 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. While 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, stated above, 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 (CMS) 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, CMS 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

4.1 Overview

The cumulative TTI curve (TTI-CDF) was introduced in Section 1.1.1 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/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.

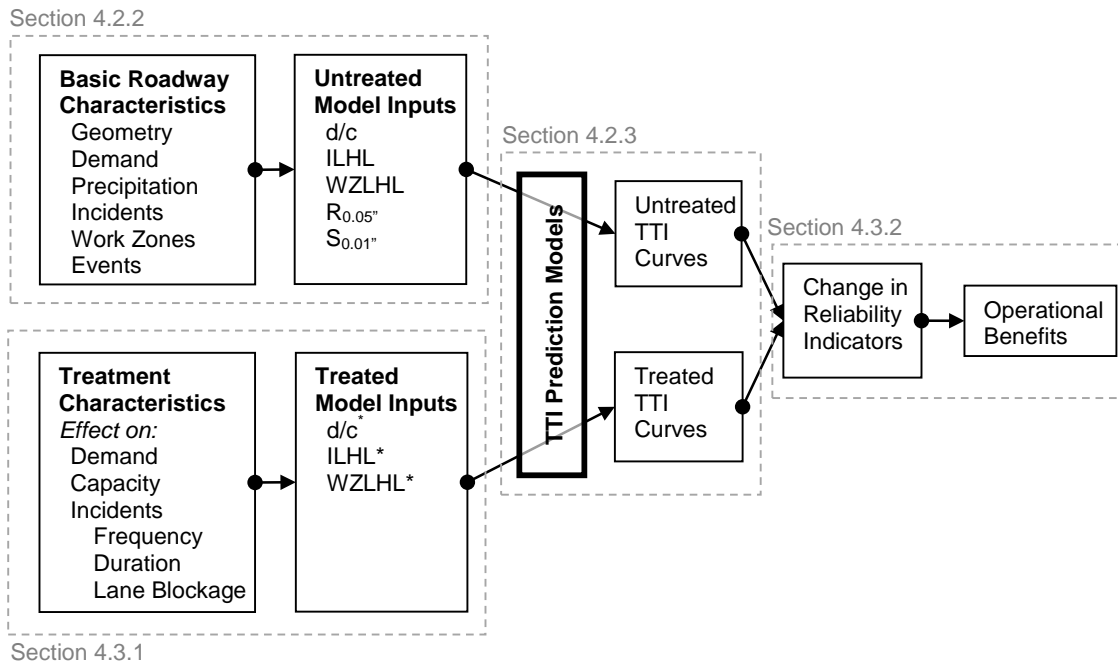


Figure 4–1. Calculation of Operational Effectiveness

4.2 Prediction of the Cumulative TTI Curve (TTI-CDF)

4.2.1 Background

Research by Margiotta et al. 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:

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

The Project L03 models were developed for several time-slices (peak-hour, peak-period, mid-day, 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 hours of the day. This approach provides the capability to consider all incidents or events that may potentially result in nonrecurrent congestion (not just those that occur during already congested periods), and to aggregate hourly operational measures into meaningful daily and annual statistics that can be used in economic analysis. Only one set of Project L03 models is based on a single-hour time-slice: the peak-hour models. The Project L07 research team revised and extended these models to apply to non-peak (uncongested) hours as well.

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

4.2.2 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.

d/c —Demand-to-Capacity Ratio

This variable gives an indication of the level of day-to-day congestion on the highway facility, and is defined as follows:

d/c : *the ratio of demand(d) to capacity(c) for a given highway segment over a given time-slice.*

Calculating Demand: Demand, d , is defined as weekday non-holiday demand during the 30th-highest hour of the year during a given time-slice. Demand differs from volume in that demand represents all motorists that would travel on a section given unconstrained capacity during a given period (everyone who *wanted* to travel on the freeway section), while volume is equal to the observed or counted vehicles during the same period (everyone who *actually* traveled on the freeway section). Therefore, when demand is less than capacity, volume equals demand. There are several methods to compute demand:

1. If the analyst has access to observed volume data for each non-holiday weekday hour for the entire year (roughly 250 counts for each of the 24 hours), the analyst can directly select the 30th-highest volume (v_{30}) for each of the 24 hours. For all uncongested periods, demand equals volume (v_{30}). For periods where demand may exceed capacity, volumes can be converted to demand using one of the following two methods:
 - a. If volume and speed data are available in 5-minute increments, the analyst can use the method developed in SHRP 2 Project L03 to compute demand (2). The procedure identifies consecutive 5-minute periods during which the mean speed drops below a congested level (typically the 35- to 45-mph range), and estimates demand 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-minute increments, and the analyst merely has 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 estimate or measure the evolution of the queue during that congested period. The total 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:

$$d = v_{30} + q \tag{8}$$

2. If, as is often the case, the analyst has a single- or multi-day count, the following procedure can be used to compute the volume for the 30th-highest hour. Most state DOTs tabulate factors that allow conversion of ADT to 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:

$$AADT = ADT \times f_{\text{month}, m} \times f_{\text{day}, d} \tag{9}$$

where: $f_{\text{month}, m}$ and $f_{\text{day}, d}$ 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 using the following equation:

$$v_{30} = v_{obs} \left(\frac{f_{month,MAX} f_{day,AVG}}{f_{month,m} f_{day,d}} \right) \quad (10)$$

where: $f_{month,MAX}$ represents the factor for the *maximum* month of the year and $f_{day,AVG}$ represents the *average* of the factors for all 5 weekdays. Thus, this equation 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 the 30th-highest hour is higher than the equation above would suggest, due to extreme volume fluctuations or the presence of major traffic generators. One way to accommodate this 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. **NOTE:** Because the frequency of events and demand surges varies from facility to facility, and city to city, incorporating event-related demand surges into reliability calculations is a complex endeavor that has not been fully addressed in previous research. The above method for including events is recommended as an initial procedure.

To convert v_{30} to demand (d) for use in the d/c equation, procedure (b) above using field-observations ($d = v_{30} + q$) is recommended for any periods that experience congestion.

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/daily factors described above may also need to be applied.

All demand volumes should be converted to passenger-car equivalents using heavy vehicle percentages and PCE factors from HCM Chapter 11 (2).

Calculating Capacity: To calculate capacity (c), procedures from Chapter 11 of the HCM 2010 are used to derive the free-flow speed for the freeway section 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 (veh/h). It should be noted, however, that 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-hour 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 makes a distinction between short- and long-term work zones. Short-term work zones (lasting seven 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, and this capacity should be used to test against any potential improvements affecting nonrecurrent congestion in the work zone (such as emergency pull-offs, etc.). 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 Section 3.10.3; 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 hours of the day.

LHL—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. It is defined as follows:

LHL: *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 are defined and described below.

ILHL—Lane-Hours Lost Due to Incidents

ILHL: *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 follows:*

$$ILHL = \frac{1}{60} \sum_i (N_{incidents,i} N_{blocked,i} T_{incidents,i}) \quad (11)$$

where: $N_{incidents,i}$ = number of incidents of type i during the time-slice
 $N_{blocked,i}$ = average number of lanes blocked per incident of type i
 $T_{incident,i}$ = average duration of incident of type i , min

Each element of the ILHL equation is discussed below.

Incident Type, i: 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/fatal
- Disabled vehicle—nonlane-blocking (shoulder)
- Disabled vehicle—lane-blocking
- Other noncrash incidents

Many items can potentially be included in the “other noncrash incidents” category, including roadway obstructions, message-board gawking, etc. The Project L07 research team also 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 our analysis methodology is applicable for evaluating treatments that mitigate this type of gawking.

Calculating $N_{incidents}$: Calculating $N_{incidents, i}$ (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 is unavailable, the values in Table 4-1 are suggested as defaults. The first two values in the table are based on Project L03 research (I), and the relative proportions of noncrash incidents are based on Project L07 discussions with highway agencies.

Table 4-1. Suggested (Default) Proportions for Noncrash Incidents

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

Diurnal Distribution of $N_{incidents}$: Since $N_{incidents, i}$ must be calculated for each hour of the day, the analyst must distribute annual incidents over 24 hours. 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 has 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-hour period.

Using methods discussed in HCM Chapter 11 (see HCM Exhibit 11-3), the average operating speed, S , for each hour is calculated based on the hourly vehicular volume (or demand), V , and free-flow speed. The density, D , for each hour i can then be determined:

$$D_i = \frac{V_i}{S_i} \quad (12)$$

Using this density, the analyst can then use the L07 crash-density relationship presented in Table 4-2 to predict a crash rate (per MVT) for that hour of the day for each crash type.

Table 4-2. Predicted Crash Rate as a Function of Traffic Density (Project L07)

Crash severity	Crash rate (as function of Density, D_i)					
	If $D_i < 20$	If $20 \leq D_i \leq 78$				If $D_i > 78$
		$C = a_1 D_i^3 + a_2 D_i^2 + a_3 D_i + a_4$				
		a_1	a_2	a_3	a_4	
Fatal or major injury, $C_{fatal\ or\ major}$	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.

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, $N_{incidents,i}$ is calculated for each hour of the day, i , as follows:

$$N_{incidents,i} = \frac{C_{H,i}^*}{\sum_{j=1}^{24} C_{H,j}^*} (C_D) \quad (13)$$

- 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)
 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/type of outside barriers. As these methods become more widely adopted, analysts can use them, coupled with procedures from the AASHTO *Highway Safety Manual* (HSM), 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:

$$N_{incidents,i} = \frac{V_{H,i}}{\sum_{j=1}^{24} V_{H,j}} (I_D) \quad (14)$$

- where: $V_{H,i}$ = traffic volume for hour i
 I_D = daily incident total for given incident type (see default percentages in Table 4-1)

Calculating $N_{blocked,i}$: To calculate $N_{blocked,i}$ (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:

$$N_{blocked,i} = N_L (1 - R_{cap,i}) \quad (15)$$

- where: $N_{blocked,i}$ = average number of lanes blocked per incident of type i

- N_L = number of lanes on the facility (one direction)
 $R_{cap,i}$ = capacity for incident of type i (see Table 4-3 and text below)

To calculate $R_{cap,i}$, 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_{cap} as well as the assumptions used to develop them from the HCM.

Table 4-3. $R_{cap,i}$ Values Used to Calculate $N_{blocked,i}$ for ILHL

No. freeway lanes (one direction)	Crashes			Non-crash incidents (disabled vehicle)		
	PDO	Minor injury	Major injury & 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
<i>Values above are adapted from HCM Exhibit 10-17, based on assumed conversions below from blockage type to incident type</i>						
<i>Shoulder Disablement</i>	<i>0%</i>	<i>0%</i>	<i>0%</i>	<i>100%</i>	<i>0%</i>	<i>50%</i>
<i>Shoulder Crash</i>	<i>72%</i>	<i>59%</i>	<i>5%</i>	<i>0%</i>	<i>0%</i>	<i>39%</i>
<i>1 Lane Blocked</i>	<i>26%</i>	<i>28%</i>	<i>35%</i>	<i>0%</i>	<i>96%</i>	<i>10%</i>
<i>2 Lanes Blocked</i>	<i>2%</i>	<i>10%</i>	<i>45%</i>	<i>0%</i>	<i>3%</i>	<i>1%</i>
<i>3 Lanes Blocked</i>	<i>0%</i>	<i>3%</i>	<i>15%</i>	<i>0%</i>	<i>1%</i>	<i>0%</i>

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

Table 4-4. Incident Duration Default Values, $T_{incident}$ (minutes)

Noncrash Incidents		Crash Incidents	
Lane-blocking	20	Property damage only	28
Nonlane-blocking	26	Minor injury	40
Other noncrash incidents	28	Major injury and fatal	45

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 prior to 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 can be inferred from the figure, the timing of several events—including responder notification (via 911, etc.), 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.

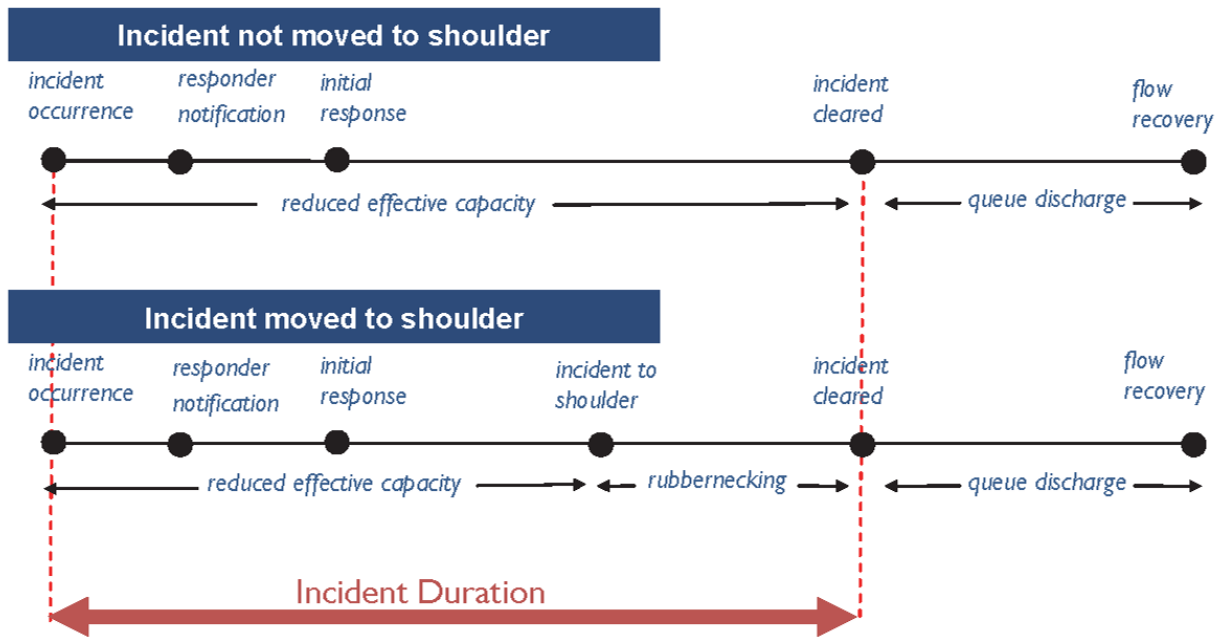


Figure 4–2. Typical Incident Timelines

WZHL—Lane-Hours Lost Due to Work Zones

WZLHL: 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 follows:

$$WZLHL = \left(1 - \frac{c_{wz} N_{lanes,WZ}}{c N_{lanes}} \right) N \text{ days} \quad (16)$$

where: c_{wz} = per-lane capacity of the work zone (passenger cars per hour per lane). The 2010 HCM (Chapter 10) suggests a default capacity of 1,600 pcphpl, with adjustments due to lane width and ramp presence.
 $N_{lanes,WZ}$ = number of open lanes through the work zone.

- c = per-lane capacity of the freeway section prior to 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 prior to establishment of the work zone.
- N_{days} = number of days the work zone is active during the time-slice

For the purposes of Project L07, long-term work zones are 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).

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.

$R_{0.05}$ —Hours of Rainfall Exceeding 0.05 in

$R_{0.05}$: For a particular time-slice, the total number of hours in which 0.05 inches or more of rainfall was 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 this data based on 10 years (2001 through 2010) of hourly precipitation data at 387 weather stations across the U.S.—see Figure 4–3. The spreadsheet tool described in Section 2.3 incorporates the rainfall database to automatically determine a value for $R_{0.05}$ when any city in the U.S. is selected.

$S_{0.01}$ —Hours of Snowfall Exceeding 0.01 in

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

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



Figure 4-3. U.S. Weather Stations Used to Determine $R_{0.05''}$ and $S_{0.01''}$
 (©Microsoft Streets and Trips)

4.2.3 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 developed in Project L03, and enhanced in Project L07, 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 the Appendix.

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:

$$TTI_n = \begin{cases} TTI_{NP,n} \times e^{(c_n R_{0.05''} + d_n S_{0.01''})} & \text{for } d/c \leq 0.8 \\ \frac{TTI_{NP,n}}{N_{days}} \times \left[N_{NP} + V_{FF} \left(\frac{R_{0.05''}}{c1_n V_{FF} + c2_n TTI_{NP,n}} + \frac{S_{0.01''}}{d1_n V_{FF} + d2_n TTI_{NP,n}} \right) \right] & \text{for } d/c > 0.8 \end{cases} \quad (17)$$

- where:
- TTIn = the predicted n th-percentile travel-time index
 - $TTI_{NP,n}$ = the non-precipitation portion of $TTI_n = e^{(a_n d/c + b_n LHL)}$
 - LHL = lane-hours lost due to incidents and work zones (see Section 4.2.2)
 - d/c = demand-to-capacity ratio (See Section 4.2.2)
 - $R_{0.05''}$ = number of hours in time-slice with rain exceeding 0.05 in (See Section 4.2.2)
 - $S_{0.01''}$ = number of hours in time-slice with snow exceeding 0.01 in (See Section 4.2.2)
 - Ndays = number of hours in time-slice (365)
 - NNP = number of hours in time-slice with no precipitation = Ndays – $R_{0.05''}$ – $S_{0.01''}$
 - VFF = free-flow travel time on segment, mph

- an, bn = nth-percentile coefficients for non-precipitation components (d/c and LHL). (See Table 4-5)
- cn, dn = nth-percentile coefficients for rain and snow components, respectively (d/c < 0.8). (See Table 4-5)
- c1n, c2n = nth-percentile coefficients for rain component (d/c > 0.8). (See Table 4-5)
- d1n, d2n = nth-percentile coefficients for snow component (d/c > 0.8). (See Table 4-5)

Table 4-5. TTI Prediction Model Coefficients

N (percentile)	d/c ≤ 0.8 ^a				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

^a Coefficients for d/c ≤ 0.8 are continuous functions of n. See text below for more description.

For the d/c ≤ 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 sub-coefficients, as shown in the equation below (with values in Table 4-6).

$$coeff_n = wn + xy^z(n-1) \tag{18}$$

- where: coeff_n = one of the four coefficients in the TTI_n formula (an, bn, cn, dn)
- n = percentile (scaled between 0 and 1.0)
- w,x,y,z = sub-coefficient (shown in Table 4-6)

Table 4-6. Sub-Coefficient Values for TTI Prediction Model (d/c < 0.8)

coeff _n	Sub-coefficients			
	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

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

The preceding section included a detailed explanation of methods to construct a predictive cumulative TTI curve based on four primary variables (d/c, LHL, R_{0.05}, and S_{0.01}). Chapter 1 of this report described 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. Not all treatments studied in Project L07 are discussed in this section, for various reasons:

- 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 it 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.

4.3.1 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 follows:

$$\text{Untreated: } TTI = f\{d/c, ILHL, WZLHL, R_{0.05}, S_{0.01}\} \quad (19)$$

$$\text{Treated: } TTI^* = f\{d/c^*, ILHL^*, WZLHL^*, R_{0.05}^*, S_{0.01}^*\} \quad (20)$$

where: f = a mathematical function (as described in Section 4.2.3)
 $(^*)$ = 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 (d/c)

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 of these treatments have the same general effect—they increase the c term in the denominator of d/c:

$$d/c^* = \frac{d}{c^*} = \frac{d}{rc} \quad (21)$$

where: c^* = treated capacity
 c = original capacity
 r = ratio between the two (c^*/c)
 d = demand (here assumed unchanged)
 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, $r = f_{LW}^*/f_{LW}$, the ratio of the treated and untreated HCM lane width adjustment factors for the respective widths. Similarly, shoulder addition or widening is based on the HCM adjustment factor for lateral clearance, f_{LC} .

Case IB: Demand Reductions

Demand-reduction strategies could include construction of alternate routes, relief of bottlenecks on existing alternate routes, or introduction of HOV lanes (which have more complex effects beyond pure demand reduction). Case IB strategies affect the numerator of d/c :

$$d/c^* = \frac{d^*}{c} = \frac{rd}{c} \quad (22)$$

where: d^* = demand after the strategy is implemented
 d = original demand
 r = ratio between the two (d^*/d)
 c = capacity (here assumed unchanged)
 d/c^* = resulting demand-to-capacity ratio

Class II: Incident Lane-Hours Lost (ILHL)

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 Section 4.2.2), as shown below.

$$ILHL_i^* = \frac{N_{incidents,i}^* N_{blocked,i}^* T_{incidents,i}^*}{60} \quad (23)$$

This class is further subdivided into six cases, as described below. These cases are not necessarily exhaustive, but cover the relevant nonrecurrent congestion design treatments studied and provide a guide that could be extrapolated to other types of ILHL-reducing treatments.

NOTE: In all Cases, $N_{blocked,i} = R_{cap,i} N_{lanes}$ except where noted. See Table 4-3 for appropriate values of $R_{cap,i}$.

Case IIA: Incident Elimination, Average Treatable Incident Duration Unspecified

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 incident for which the treatment will be applied. In this case, only one variable is affected:

$$N_{incidents,i}^* = (1 - p_i)N_{incidents,i} \quad (24)$$

It should be noted that the $(1-p_i)$ term is directly related to the concept of a Crash Modification Factor. Formally introduced to practice through the AASHTO *Highway Safety Manual* (HSM) (4), CMFs can be defined as the ratio of the expected average crash frequency in a treated condition to the expected frequency in the untreated condition. Since the frequency is defined as the number of incidents over a specified period, the following logic applies:

$$N_{incidents,i}^* = CMF \times N_{incidents,i} = (1 - p_i)N_{incidents,i} \Rightarrow p_i = 1 - CMF \quad (25)$$

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 ($N_{blocked,i}^* = N_{blocked,i}$ and $T_{incidents,i}^* = T_{incidents,i}$). Treatments in this category include Wildlife-Vehicle Collision Reduction, Anti-Icing, Snow Fences, and Blowing Sand Reduction.

Case IIB: Incident Elimination, Average Treatable Incident Duration Specified

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 ($T_{incidents,i}$) in the base condition is already specified, the analyst must specify $T_{treatable}$, 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:

$$T_{incidents,i}^* = \frac{T_{incidents,i} - p_i T_{treatable}}{1 - p_i} \quad (26)$$

With a notable boundary condition:

$$T_{treatable} \leq \frac{T_{incidents,i}}{p_i} \quad (27)$$

$N_{incidents,i}^*$ is calculated as in Case IIA (including the same relationship with CMFs), and $N_{blocked,i}^*$ remains equal to the untreated condition ($N_{blocked,i}^* = N_{blocked,i}$). 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 with 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 below.

$$\text{Unaffected incidents: } ILHL^*_1 = (1-p_i) N_{\text{incidents},i} N_{\text{blocked},i} T_{\text{incidents},i}/60 \quad (28)$$

$$\text{Affected incidents: } ILHL^*_2 = p_i N_{\text{incidents},i} N_{\text{blocked},i} T_i^*/60 \quad (29)$$

The total treated $ILHL$ is the sum of these two terms:

$$ILHL^* = ILHL^*_1 + ILHL^*_2 \quad (30)$$

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, Average Treatable Incident Duration Unspecified

Case IID includes treatments which essentially transform a portion of incidents, mid-duration, 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 prior to 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, as shown below.

$$\text{Unaffected incidents: } ILHL^*_1 = (1-p_i) N_{\text{incidents},i} N_{\text{blocked},i} T_{\text{incidents},i}/60 \quad (31)$$

$$\text{Affected incidents, pre-conversion: } ILHL^*_2 = p_i N_{\text{incidents},i} N_{\text{blocked},i} T_i^*/60 \quad (32)$$

$$\text{Affected incidents, post-conversion: } ILHL^*_3 = p_i N_{\text{incidents},i} N_{\text{blocked},k} (T_{\text{incidents},i} - T_i^*)/60 \quad (33)$$

The total treated $ILHL$ is the sum of these three terms:

$$ILHL^* = ILHL^*_1 + ILHL^*_2 + ILHL^*_3 \quad (34)$$

NOTE: 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 non-blocking shoulder incident). Treatments in this category include Accessible Shoulder, Alternating Shoulder, Crash Investigation Site, and Emergency Pull-off.

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

Like Case IID, Case IIE includes treatments that essentially transform a portion of incidents, mid-duration, 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 below.

Unaffected incidents:

$$ILHL^*_1 = (1-p_i) N_{\text{incidents},i} N_{\text{blocked},i} (T_{\text{incidents},i} - p_i T_{\text{treatable},i}) / [60 \times (1-p_i)] \quad (35)$$

Affected incidents, pre-conversion:

$$ILHL^*_2 = p_i N_{\text{incidents},i} N_{\text{blocked},i} T_i^* / 60 \quad (36)$$

Affected incidents, post-conversion:

$$ILHL^*_3 = p_i N_{\text{incidents},i} (1-p_r) N_{\text{blocked},k} (T_{\text{treatable},i} - T_i^*) / 60 \quad (37)$$

The total treated ILHL is the sum of these three terms:

$$ILHL^* = ILHL^*_1 + ILHL^*_2 + ILHL^*_3 \quad (38)$$

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 address 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 analysis of diversion-related treatments:

- The “capacity,” or throughput, of the diversion treatment itself, 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, $T_{treatable}$. As with Cases IIB and IIE, it is assumed that incidents for which this treatment would be deployed are longer than average. $T_{treatable}$ is used in a different way with Case IIF from the way it is used in Cases IIB and IIE, as discussed below.

For Case IIF, the reduction in lane-hours lost is treated as “lane-hours gained”:

$$\Delta ILHL = \frac{p_i N_{incidents,i} N_{lanes} c_{div} T_{treatable}}{C} \quad (39)$$

$$ILHL^* = ILHL + \Delta ILHL \quad (40)$$

Therefore, unlike other cases, ILHL isn’t made up of three terms, although $\Delta ILHL$ is made up of three terms analogous to the typical ILHL calculation:

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

Treatments in this class include Emergency Crossovers, Controlled/Gated Turnarounds, Driveable Shoulder, and Movable Cable Median Barrier. Table 4-7 summarizes the ILHL equation terms for each of the six cases and their sub-cases.

Table 4-7. Terms in Treated ILHL Equations: Six Cases

Case ¹	$N_{incidents,i}^*$	$N_{blocked,i}^*$	$T_{incidents,i}^*$
IIA: Incident Elimination, Average Treatable Incident Duration Unspecified	$(1-p_i)N_{inc,i}$	$(1-R_{cap,i})N_{lanes}$	$T_{inc,i}$
IIB: Incident Elimination, Average Treatable Incident Duration Specified	$(1-p_i)N_{inc,i}$	$(1-R_{cap,i})N_{lanes}$	$(T_{inc,i} - p_i T_{treatable}) / (1-p_i)$
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, Average Treatable Incident Duration Unspecified, Passive Treatment	unaffected: $(1-p_i)N_{inc,i}$	$(1-R_{cap,i})N_{lanes}$	$T_{inc,i}$
	affected, pre-conversion: $p_i N_{inc,i}$	$(1-R_{cap,i})N_{lanes}$	T_i
	affected, post-conversion: $p_i N_{inc,i}$	$(1-R_{cap,k})N_{lanes}$	$T_{inc,i} - T_i^*$
IIe: Incident Type Conversion, Average Treatable Incident Duration Specified, Active Treatment	unaffected: $(1-p_i)N_{inc,i}$	$(1-R_{cap,i})N_{lanes}$	$(T_{inc,i} - p_i T_{treatable}) / (1-p_i)$
	affected, pre-conversion: $p_i N_{inc,i}$	$(1-R_{cap,i})N_{lanes}$	T_i
	affected, post-conversion: $p_i N_{inc,i}$	$(1-R_{cap,i})(1-p_r)N_{lanes}$	$T_{treatable} - T_i^*$
IIF: Incident Diversion	$ILHL^* = ILHL + \frac{p_i N_{incidents,i} N_{lanes} C_{div} T_{treatable}}{C}$		

¹ For Cases A-E, $ILHL^* = N_{incidents,i}^* \times N_{blocked,i}^* \times T_{incidents,i}^*$. For Case F, $ILHL^*$ is as shown in the table.

Class III: WZLHL

For short-term work zones, there are three variables that could 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 follows:

$$WZLHL^* = \left(1 - \frac{C_{WZ} N_{lanes,WZ}^*}{C N_{lanes}^*} \right) N_{days}^* \quad (41)$$

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 $R_{0.05}$ ", since they cannot influence the amount of rain that falls. However, the variable $R_{0.05}$ " is an important variable in the model because it helps describe the base conditions.

Class V: $S_{0.01}$ "

Similar to the discussion of the variable $R_{0.05}$ ", design treatments do not affect the variable $S_{0.01}$ ". However, the variable $S_{0.01}$ " is an important variable in the model because it helps describe

Table 4-8. Suggested Default Coefficients/Terms for Treatment Mapping to TTI Prediction Models (Italicized values are user-modifiable in the analysis tool)

Treatment	Class/Case	Portion of incidents using/ affected by treatment, p_i					
		Crashes			Non-crashes		
		PDO	Min	Maj/Fat	NLB	LB	Other
Anti-icing systems	IA	<i>0.10</i>	<i>0.10</i>	<i>0.10</i>	na	na	na
Blowing sand	IA	<i>0</i>	<i>0</i>	<i>0</i>	na	na	na
Extra high median barrier	IA	Apply to opposite-direction incidents					
Snow fence	IA	<i>0.10</i>	<i>0.10</i>	<i>0.10</i>	na	na	na
Wildlife collision reduction	IA	*	*	*	na	na	*
Runaway truck ramp	IB	<i>0.001</i>	<i>0.001</i>	<i>0.001</i>	na	na	na

		Portion of incidents using/ affected by treatment, p_i					
		Crashes			Non-crashes		
		PDO	Min	Maj/Fat	NLB	LB	Other
Emergency access between interchanges	IC	<i>0.05</i>	<i>0.10</i>	<i>0.20</i>	na	na	na
Accessible shoulder	ID	<i>0.50</i>	<i>0.30</i>	<i>0.10</i>	na	<i>0.60</i>	<i>0.25</i>
Alternating shoulder	ID	<i>0.35</i>	<i>0.25</i>	<i>0.05</i>	na	<i>0.50</i>	<i>0.20</i>
Crash investigation site	ID	<i>0.40</i>	<i>0.20</i>	<i>0</i>	<i>0.20</i>	<i>0.40</i>	<i>0.10</i>
Emergency pull-off	ID	<i>0.40</i>	<i>0.20</i>	<i>0</i>	na	<i>0.15</i>	<i>0.10</i>

		Incident duration with treatment, T_i (min)					
		Crashes			Non-crashes		
		PDO	Min	Maj/Fat	NLB	LB	Other
		5	5	5	na	na	na
		25	35	45	na	20	20
		25	35	45	15	20	20
		25	35	45	15	20	20
		25	35	45	15	20	20

		Ratio of applicable duration to average incident length			% lost capacity restored, $p_{r,i}$		
		PDO	Min	Maj/Fat	PDO	Min	Maj/Fat
		2	2	2	10	10	10

		Deployment Time, min					
Incident screens	IE	<i>0</i>	<i>0.05</i>	<i>0.10</i>	20		

		Applicable duration, hrs			v/c threshold of application		
		PDO	Min	Maj/Fat	PDO	Min	Maj/Fat
Emergency crossovers	IF	<i>0</i>	<i>0.01</i>	<i>0.05</i>	na	na	na
Controlled/gated turnarounds	IF	<i>0</i>	<i>0.01</i>	<i>0.05</i>	na	na	na
Driveable shoulder	IF	<i>0.05</i>	<i>0.15</i>	<i>0.25</i>	na	<i>0.05</i>	<i>0.05</i>
Movable cable median barrier	IF	<i>0</i>	<i>0.01</i>	<i>0.05</i>	na	na	na

* There are many varieties of wildlife collisions, each with its own set of potential treatments and effects.
na = not applicable.

the base conditions. While 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.

4.3.2 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 reducing 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.

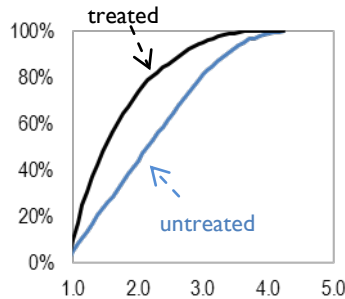


Figure 4–4. Comparison of Treated and Untreated TTI Curves

Figure 4–5, a version of which was presented in Chapter 4.1, is re-presented here to illustrate the process that leads to the final calculation of operational benefits.

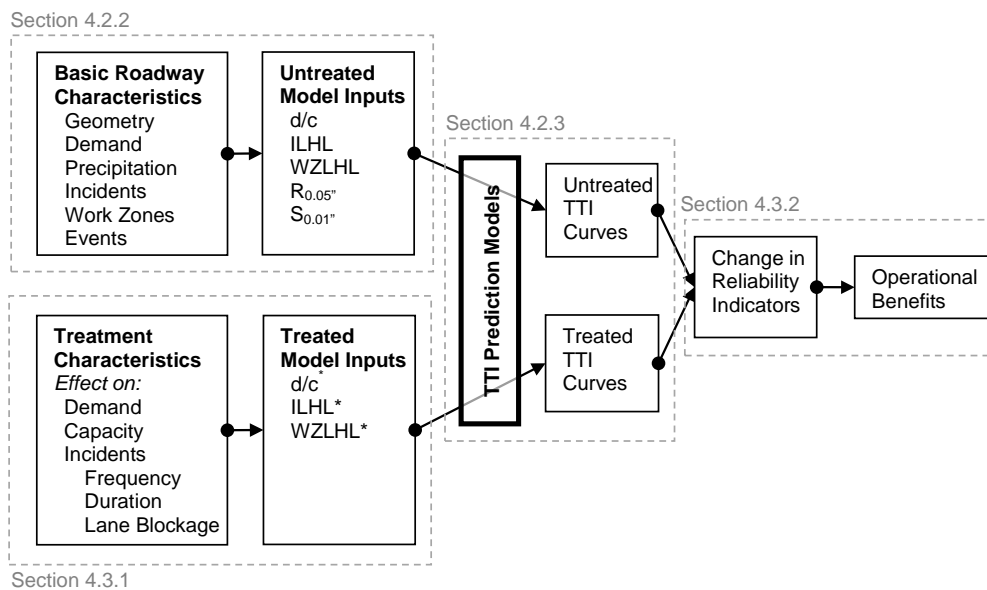


Figure 4–5. Calculation of Operational Effectiveness

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 and the standard deviation. Calculation of changes in these measures is discussed in Sections 4.3.3 and 4.3.4, respectively. Calculation of other measures, such as the semi-variance and various indices, is discussed in Section 1.1.

The TTI prediction models have a feature that is important to note in reliability calculations: there is not a 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.

4.3.3 Change in Lateness Index

As described in Chapter 1, the unitless area between the cumulative TTI curve and the vertical line at $TTI=1.0$, termed the Lateness Index, is a measure similar to the mean of the TTI distribution. The difference between untreated and treated Lateness Indices 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.

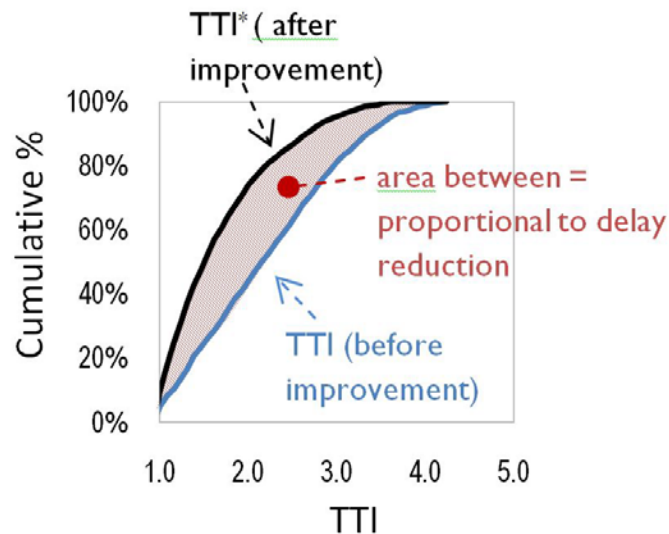


Figure 4–6. Change in Lateness Index

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} :

$$TT_{FF} = \frac{L}{S_{FF}} \quad (42)$$

Percentiles of the TTI curve can be determined from the TTI prediction models described in Section 4.2.3:

$$TTI_n = \begin{cases} TTI_{NP,n} \times e^{(c_n R_{05} + d_n S_{01})} & \text{for } d/c \leq 0.8 \\ \frac{TTI_{NP,n}}{N_{days}} \times \left[N_{NP} + V_{FF} \left(\frac{R_{05}}{c1_n V_{FF} + c2_n TTI_{NP,n}} + \frac{S_{01}}{d1_n V_{FF} + d2_n TTI_{NP,n}} \right) \right] & \text{for } d/c > 0.8 \end{cases} \quad (43)$$

The actual travel time TT corresponding to any given value of TTI is:

$$TT = (TTI)(TT_{FF}) = TTI \left(\frac{L}{S_{FF}} \right) \quad (44)$$

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

$$\Delta TT_i = TT_i - TT_i^* = (TTI_i - TTI_i^*) \frac{L}{S_{FF}} \quad (45)$$

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 = travel time index for percentile n of the cumulative TTI distribution (TTI-CDF) in the untreated condition

TTI_n^* = travel time index 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 $d/c \leq 0.8$ does predict continuous distributions), the total vehicle-hours of delay (or change in Lateness Index) for the entire time-slice could be calculated as:

$$\Delta LI_k = N_d V L S_{ff} \int_{i=0}^{100\%} (TTI_i - TTI_i^*) di = N_d V L S_{ff} \int_{i=0}^{100\%} \Delta TTI_i di \quad (46)$$

where: ΔLI_k = traffic operational delay reduction due to design treatment during time-slice k (change in Lateness Index)

N_d = the number of days in the time-slice (generally assumed as 250 non-holiday weekdays)

V = the hourly vehicular volume during the time-slice

All other variables are as described before.

However, since 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 by:

$$\Delta LI_k \approx N_d VLS_{ff} (0.200\Delta TTI_{10\%} + 0.350\Delta TTI_{50\%} + 0.225\Delta TTI_{80\%} + 0.095\Delta TTI_{95\%} + 0.020\Delta TTI_{99\%}) \quad (47)$$

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.

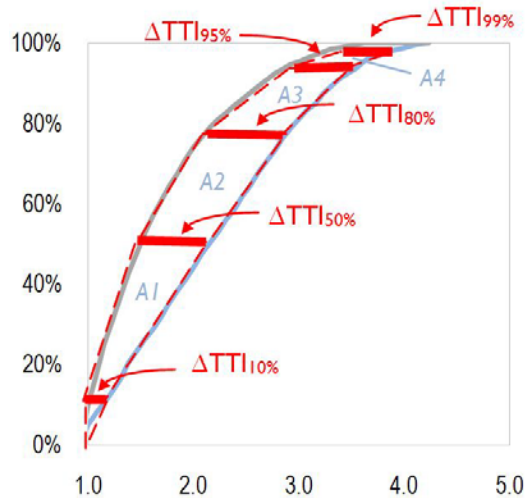


Figure 4–7. Estimating Delay by Quantifying the Area Between the Treated and Untreated TTI Curves

4.3.4 Change in Variance

Project L07 has 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 previously indicated in Chapter 1:

$$\sigma = \int_{i=0}^{100\%} (TTI_i - TTI_{mean})^2 di \quad (48)$$

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).

Since 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:

$$\sigma \approx S_{FF}(0.300\Delta_{10\%} + 0.350\Delta_{50\%} + 0.225\Delta_{80\%} + 0.095\Delta_{95\%} + 0.020\Delta_{99\%}) \quad (49)$$

where: $\Delta_n = (TTI_n - TTI_{mean})^2$

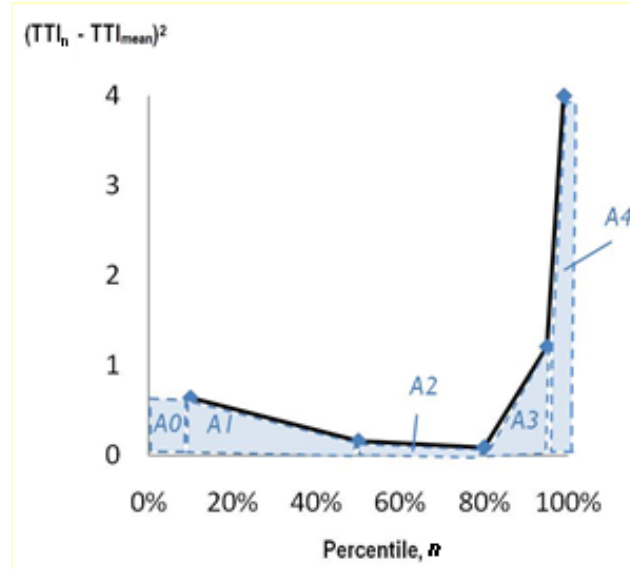


Figure 4–8. Graphical Presentation of Procedure for Approximating the Variance of the TTI Distribution

Similar approximations can be made in the case of semi-variance, σ_r :

$$\sigma_r = \int_{i=0}^{100\%} (TTI_i - 1)^2 di \quad (50)$$

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

4.3.5 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-hour 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. These are:

- 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 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.

The direct effects of design treatments on crash frequency for freeways have not been fully documented, but have recently been investigated in NCHRP Project 17-45 for inclusion in the HSM (4). 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* (RSAP) (5). The relationship between congestion and safety has been determined in Project L07 and included in the assessment of design treatments.

5.1 Direct Effects of Design Treatments on Safety

A new safety prediction methodology for freeways has been developed in NCHRP Project 17-45 (3). This methodology is currently in the approval process for inclusion in the HSM (4). 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 CMF:

$$\text{CMF} = \exp (a[W_{\text{os}} - 10]) \quad (51)$$

where: W_{os} = outside shoulder width on freeway section (ft) (range: 4 to 14 ft)
 A = regression coefficient (-0.0647 for FI crashes and 0.0000 for PDO crashes)

Table 5-1 shows CMFs for the effect of changing outside shoulder width on safety. The percent change in crashes resulting from a change in outside shoulder width can be determined from the CMFs in Table 5-1 as:

$$\text{Percent change in crashes} = (\text{CMF} - 1) \times 100 \quad (52)$$

Thus, a CMF of 1.03 corresponds to a 3-percent increase in crash frequency. A CMF of 0.97 corresponds to a 3-percent decrease in crash frequency.

**Table 5-1. CMFs for Changing Outside Shoulder Width on Freeways
(Bonneson et al., 2012)**

Inside shoulder width (ft) (before)	Inside shoulder width (ft) (after)					
	2	4	6	8	10	12
Fatal-and-injury crashes (FI)						
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 (FI)						
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

The effect of inside shoulder width on safety is represented by the following CMF:

$$\text{CMF} = \exp (a[W_{is} - 6]) \quad (53)$$

where: W_{is} = inside shoulder width on freeway section (ft) (range: 2 to 12 ft)
 A = regression coefficient (−0.0172 for FI crashes and −0.0153 for PDO crashes)

Table 5-2 shows CMFs for the effect of changing inside shoulder width on safety.

Table 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:

<u>Design treatment</u>	<u>Design parameter</u>
Accessible shoulder	Inside shoulder width Outside shoulder width
Drivable shoulder	Inside shoulder width Outside shoulder width
Alternating shoulder	Inside shoulder width Outside shoulder width

**Table 5-2. CMFs for Changing Inside Shoulder Width on Freeways
(Bonneson et al., 2012)**

Inside shoulder width (ft) (before)	Inside shoulder width (ft) (after)					
	4	6	8	10	12	14
Fatal-and-injury crashes (FI)						
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 (FI)						
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

5.2 Development of Congestion/Safety Relationship

The reduction of congestion through application of design treatments or ITS 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. And, 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. On the other hand, collision severity is clearly a function of speed, so the lower speeds on roadways during congested periods may reduce overall collision severity. This tradeoff between crash frequency and severity in congested vs. uncongested conditions had never been satisfactorily quantified. Previous research on this issue for freeway facilities has been conducted by Zhou and Sisiopiku (6) and by Hall and Pendleton (7). In particular, Zhou and Sisiopiku suggest that different crash types respond in different ways to volume-to-capacity (v/c) ratios based on hourly volumes. The research results presented below illustrated 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 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 while those in Seattle included only 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 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 summarizes the available site data, and 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.

Table 5-3. Site Distribution Characteristics for Directional Freeway Segments in Seattle and Minneapolis/St. Paul

Metro area	Number of directional lanes^a	Number of sites	Length (mi)	Number of 15-min records^b
Seattle (WA)	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 (MN)	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 HOV lanes.

^b Includes records with missing volume or speed.

5.2.1 Database Development

The original detector data collected at each station on the freeways consisted of 5-minute volume and average speed data for each travel lane; speeds or volumes were missing for some 5-minute intervals on one or more lanes. Most missing data were attributed to detector malfunctions. No set of loop detectors will function across all freeway lanes all of the time; therefore, 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 where 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 percent of the 3-year study period for Seattle and 16 percent

for Minneapolis/St. Paul. In addition, because of the unusual flow conditions, 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. While this period might have been interesting (because volumes changed dramatically on many freeway segments), the changed driving conditions were new to many drivers and Mn/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-minute volume data. These flow rates included some large fluctuations. The speed and volume data were aggregated into 15-minute intervals, which provided much more stable data. Once processed, the volume and speed data were used to determine the level of service for each 15-minute interval (discussed later in this section).

Crash data for each directional freeway segment were compiled for the same 15-minute periods as the traffic volume and speed detector data based on the reported crash date and time. The crash data, obtained through HSIS, 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 are:

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

Table 5-4 summarizes the crash data (number and percent) by collision type and severity separately for Seattle and Minneapolis/St. Paul over the 3-year period.

Table 5-4. Crash Distribution by Collision Type and Crash Severity for Freeway Sections in Seattle and Minneapolis/St. Paul

Collision type	Number (percent) of crashes by crash severity									
	Fatal		A Injury		B Injury		C Injury		PDO	
Minneapolis/St. Paul (MN)										
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 (WA)										
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)

5.2.2 LOS Calculations

Level of service was computed for each 15-minute record 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 (MPO).

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-minute period was computed from the available speed and volume data as follows:

$$D_{15} = \frac{4 V_{15} f_{HV}}{S_{15}} \quad (54)$$

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

It should be noted that Equation (54) does not include the peak-hour factor so that D_{15} is based on the actual 15-minute volume, not the highest 15-minute volume during a particular hour, as is commonly used in HCM procedures.

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

LOS	Traffic density range (pc/mi/ln)
A	0 to 11
B	11 to 18
C	18 to 26
D	26 to 35
E	35 to 45
F	45+

Since the LOS categories are quite broad, a more refined LOS categorization was used to better capture the relationship between density and crash rates. These 18 LOS categories selected are as shown in Table 5-5.

Table 5-5. LOS Categories Used in the Study

LOS	Traffic density range (pc/mi/ln)	LOS	Traffic density range (pc/mi/ln)
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+

5.2.3 Development of LOS-Crash Rate Relationships

Based on the 15-minute crash rate and traffic density data, average crash rates, expressed in crashes/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 in 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 million veh-mi of travel (MVMT) with traffic density for freeway sections in the Seattle metropolitan area. Each point represents the crash rate for all 15-minute periods of the 3-year period that falls in a particular 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 hours 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/ln (i.e., LOS D through E+). However, regression modeling has still 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.

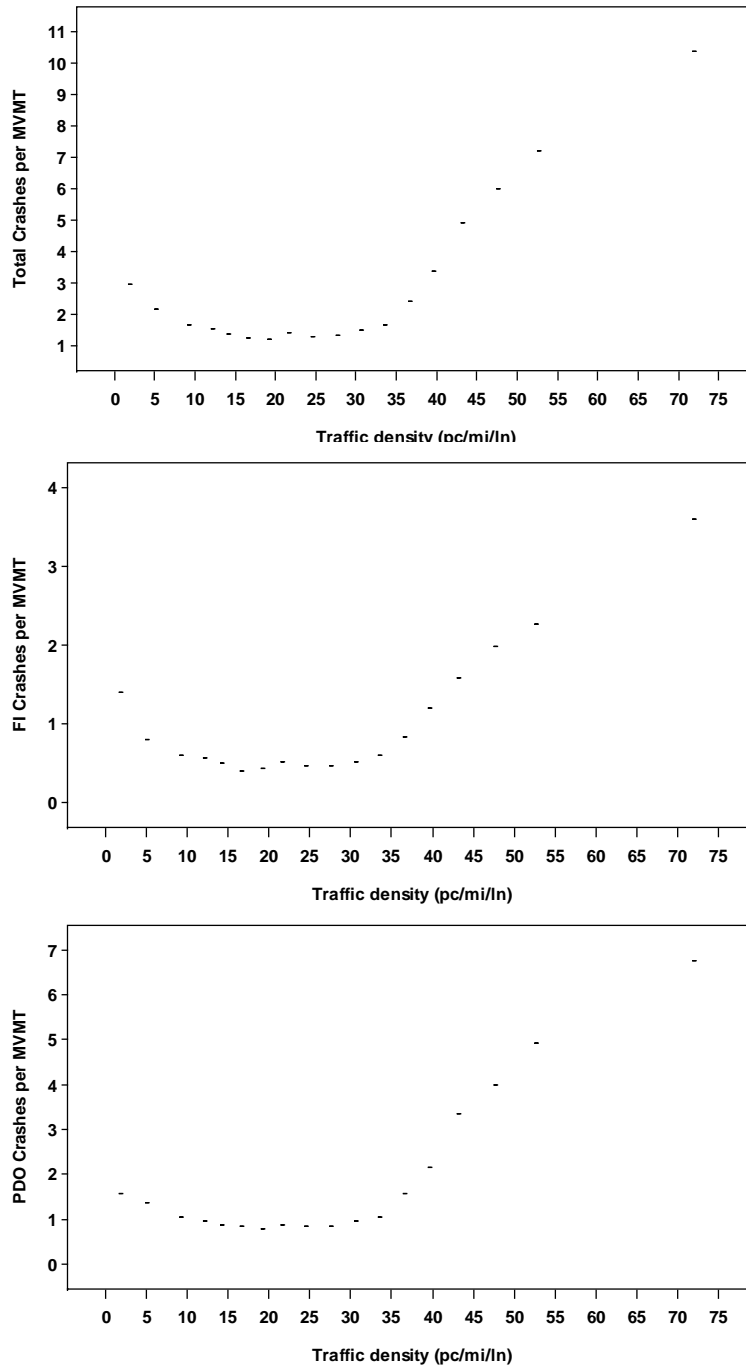


Figure 5–1. Total, FI, and PDO Crash Rates vs. Traffic Density for Freeways in the Seattle Area

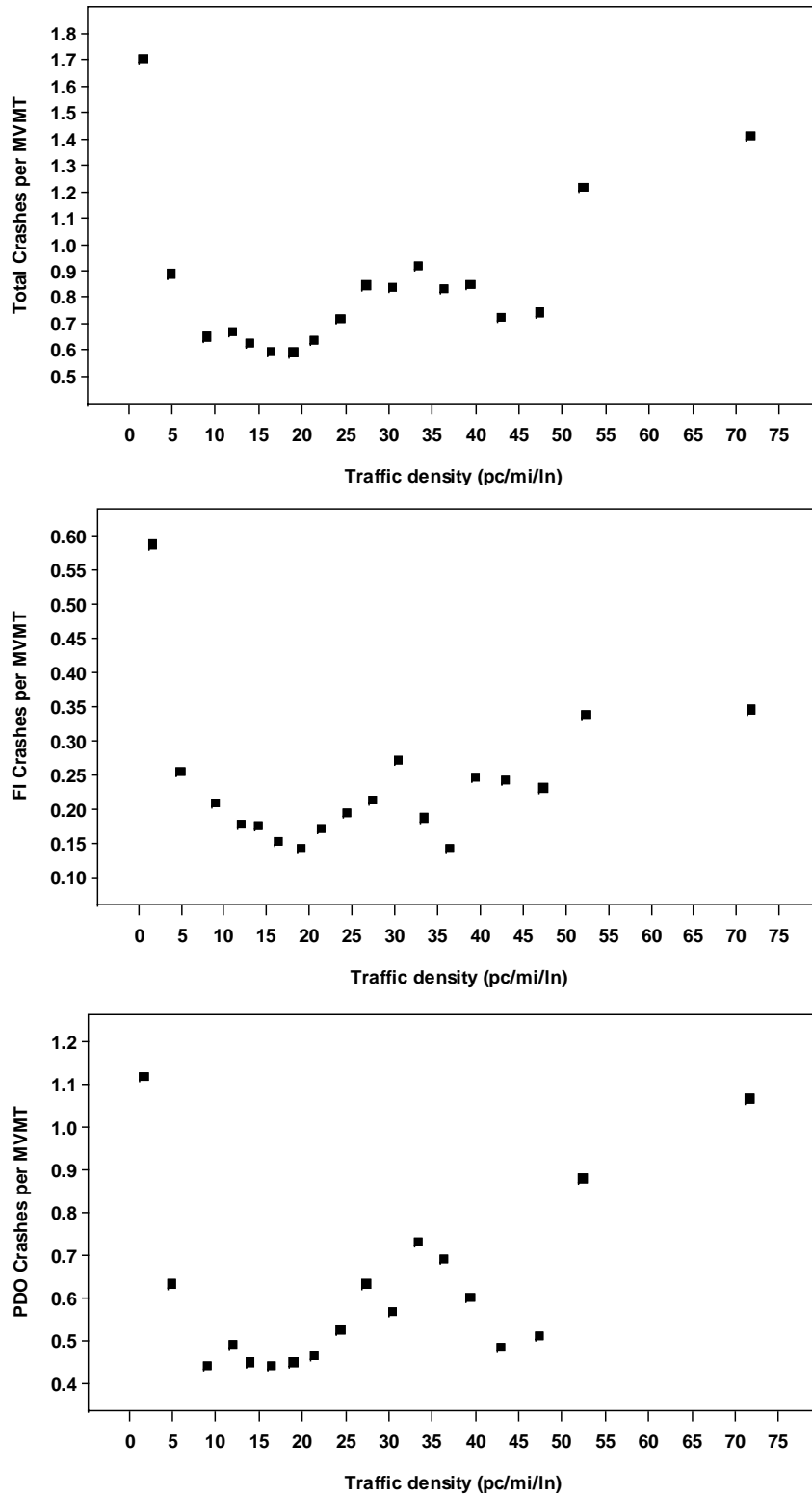


Figure 5–2. Total, FI, and PDO Crash Rates vs. Traffic Density for Freeways in the Minneapolis/St. Paul Area

Table 5-6. Crash Type Distribution for Seattle and Minneapolis/St. Paul Freeways by Level of Service Category

		Level of Service (LOS) ^a											
		A		B		C		D		E		F	
Seattle													
Crash type	Collision type	Number of crashes (percent of total)											
Single-vehicle crashes	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	(0.0)
	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
Multiple-vehicle crashes	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
Total		1,307	(100)	1,089	(100)	1,678	(100)	1,724	(100)	1,507	(100)	3,689	(100)
Minneapolis/St. Paul													
Crash type	Collision type	Number of crashes (percent of total)											
Single-vehicle crashes	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)
	RR 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
Multiple-vehicle crashes	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 Level of service assigned to each crash based on the freeway segment and the traffic conditions for the 15-min period in which the crash occurred.

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 vs. 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, fatal-and-injury (FI) crashes, and property-damage-only (PDO) crashes. Curves were fit to these data using ordinary least-squares regression analysis for the LOS range where 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 presumed 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 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 and a model for PDO crashes was obtained by subtraction.

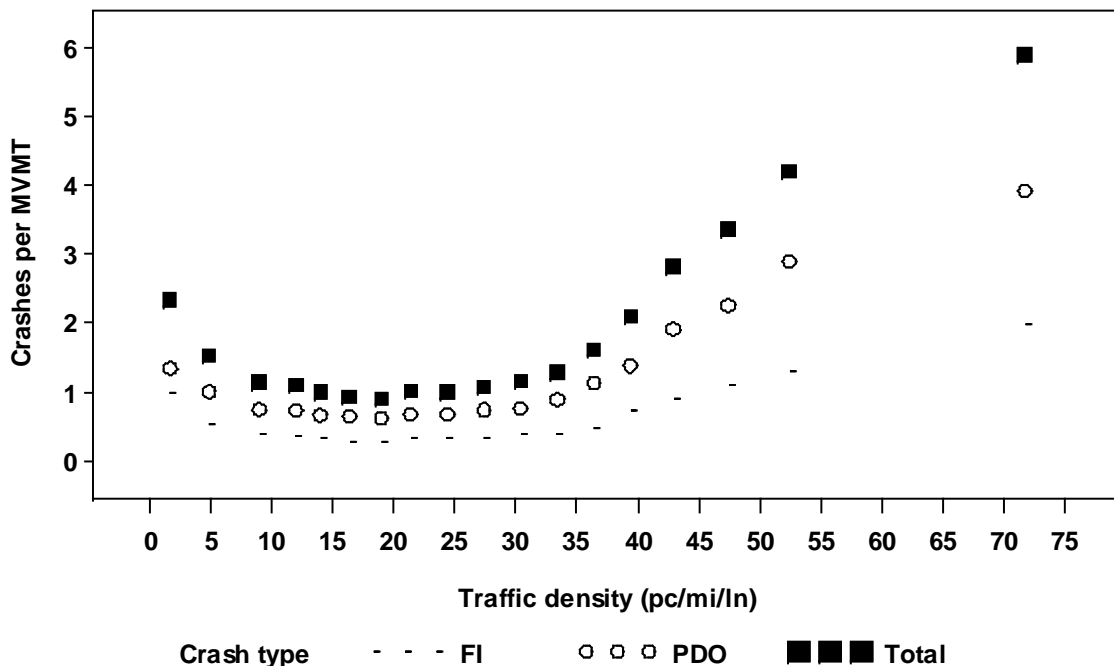


Figure 5-3. Crash Rate vs. Density for Combined Seattle and Minneapolis/St. Paul Areas

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

$$\text{Crash rate} = a_0 + a_1 \times \text{Density} + a_2 \times \text{Density}^2 + a_3 \times \text{Density}^3 \quad (55)$$

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.

Table 5-7. Regression Results for FI and Total Crash Rates vs. Density

Severity level	Regression coefficients				Model fit		Crash rate (crashes/MVMT) at specified density	
	a ₀	a ₁	a ₂	a ₃	RMSE	R ² (%)	20 pc/mi/ln	78 pc/mi/ln
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 ^a	1.614	-0.1301	0.00444	-3.01 × 10 ⁻⁵	NA	NA	0.54	4.17

^a Regression coefficients and crash rates for 20 and 78 pc/mi/ln obtained by subtraction (Total – FI).

The Total and FI curves reach a local minimum at a density around 20 pc/mi/ln. Therefore, 20 pc/mi/ln was selected as that 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/ln. The last two 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/ln). Figure 5-4 illustrates the observed and predicted crash rates as a function of traffic density. The final relationships are shown in Equations (56) through (58).

$$\text{Total crashes per MVMT} = 2.636 - 0.2143 \times D + 0.00708 \times D^2 - 4.80 \times 10^{-5} \times D^3 \quad (56)$$

$$\text{FI crashes per MVMT} = 1.022 - 0.0842 \times D + 0.00264 \times D^2 - 1.79 \times 10^{-5} \times D^3 \quad (57)$$

$$\text{PDO crashes per MVMT} = 1.614 - 0.1301 \times D + 0.00444 \times D^2 - 3.01 \times 10^{-5} \times D^3 \quad (58)$$

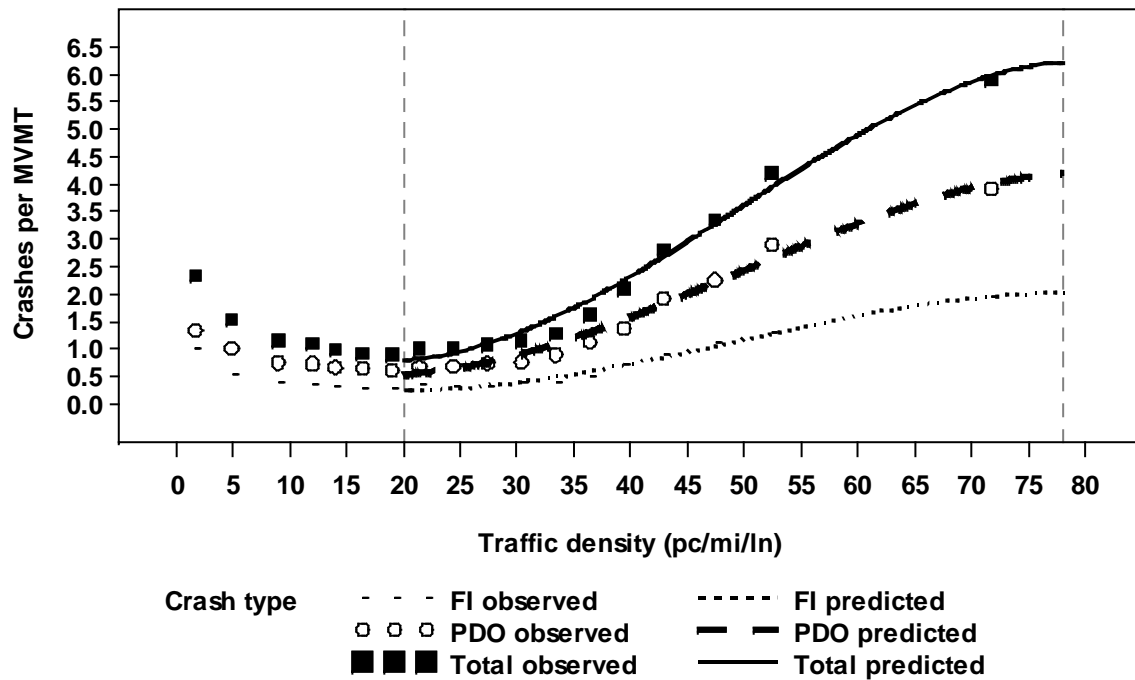


Figure 5-4. Observed and Predicted Total, FI, and PDO Crash Rates vs. Traffic Density

The crash rate-traffic density relationships shown in Figure 5-4 and Equations (56) through (58) are used in two ways in the analysis of the effectiveness of design treatments for nonrecurrent congestion. The primary application is to estimate the percent 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 based on the congestion levels present. The relationships shown in Figure 5-4 and Equations (56) through (58) are applied only in the traffic density range from 20 to 78 pc/mi/ln. Above and below this crash density range, the crash rate is assumed to be constant at the end-point values shown in the last two columns of Table 5-7. In other words, the full crash rate-traffic density relationship incorporated in the assessment methodology is:

$$\text{Total crashes per MVMT} = \begin{cases} 0.80 & \text{if Density} < 20 \text{ pc/mi/ln} \\ 2.636 - 0.2143 \times D + 0.00708 \times D^2 - 4.80 \times 10^{-5} \times D^3 & \text{if } 20 \leq \text{Density} \leq 78 \text{ pc/mi/ln} \\ 6.22 & \text{if Density} > 78 \text{ pc/mi/ln} \end{cases} \quad (59)$$

$$\text{FI crashes per MVMT} = \begin{cases} 0.25 & \text{if Density} < 20 \text{ pc/mi/ln} \\ 1.022 - 0.0842 \times D + 0.00264 \times D^2 - 1.79 \times 10^{-5} \times D^3 & \text{if } 20 \leq \text{Density} \leq 78 \text{ pc/mi/ln} \\ 2.04 & \text{if Density} > 78 \text{ pc/mi/ln} \end{cases} \quad (60)$$

$$\text{PDO crashes per MVMT} = \begin{cases} 0.54 & \text{if Density} < 20 \text{ pc/mi/ln} \\ 1.614 - 0.1301 \times D + 0.00444 \times D^2 - 3.01 \times 10^{-5} \times D^3 & \text{if } 20 \leq \text{Density} \leq 78 \text{ pc/mi/ln} \\ 4.17 & \text{if Density} > 78 \text{ pc/mi/ln} \end{cases} \quad (61)$$

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

5.3 Prediction of Crash Reduction Due to Congestion Reduction Resulting From Design Treatments

For each design treatment evaluated in Project L07, an untreated TTI curve and a treated TTI curve are predicted for each hour of the day. The Project L07 research team has devised a methodology to convert a TTI curve into an equivalent traffic density distribution, which can then utilize the safety-density relationship to predict untreated and treated crash rates. This methodology is described in the following paragraphs.

Project L03 has provided equations to predict five percentile values of the cumulative TTI distribution:

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

The lowest or zero percentile value of TTI is also known because it is, by definition, equal to 1.00.

As shown in Figure 5–5, these values can be plotted to estimate the cumulative TTI curve:

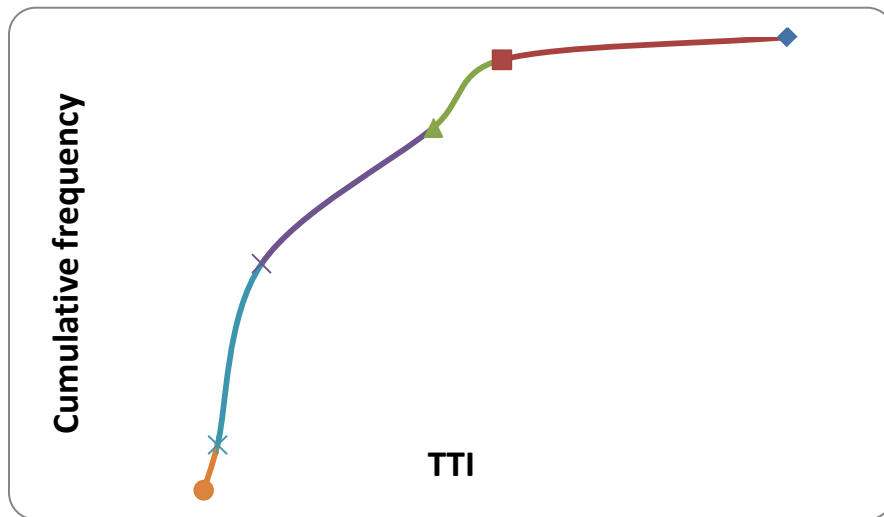


Figure 5–5. Example of Cumulative TTI Distribution Approximated From Five Percentile Values

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 (for example, 8:00 to 9:00 a.m.). These proportions are termed the “weight” of each subset, and are as follows:

- Subset1 (TTI₀-TTI₁₀): Weight₁ = 10 percent
- Subset2 (TTI₁₀-TTI₅₀): Weight₂ = 40 percent
- Subset3 (TTI₅₀-TTI₈₀): Weight₃ = 30 percent
- Subset4 (TTI₈₀-TTI₉₅): Weight₄ = 15 percent
- Subset5 (TTI₉₅-TTI₉₉): Weight₅ = 5 percent

It should be noted that Subset 5 is given the weight of 5 percent, even though the difference between 95 and 99 percent is only 4 percent. This assumption is equivalent to estimating that TTI₁₀₀ ≈ TTI₉₉.

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 using the following formula:

$$TTI_{\text{subset } i} = \frac{(TTI_{\text{lower}} + TTI_{\text{upper}})}{2} \quad (62)$$

- where:
- TTI_{subset i} = average TTI value for subset *i*
 - TTI_{lower} = lowest TTI value for the subset
 - TTI_{upper} = highest TTI value for the subset

(For example, for Subset1 (TTI₀-TTI₁₀), TTI_{lower}=TTI₀ and TTI_{upper} = TTI₁₀).

Using these values, the average travel time (TT) 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.

$$TT_{\text{subset } i} = \frac{\text{Length} \times TTI_{\text{subset } i}}{\text{FFS}} \quad (63)$$

- where:
- TT_{subset i} = average travel time for subset *i* (hr)
 - Length = length of freeway segment (mi)
 - TTI_{subset i} = average TTI value for the subset
 - FFS = free-flow speed for the freeway segment (mi/h)

The free-flow speed is determined using HCM Chapter 23 procedures [see Equation (76) later in this section]. The average speed for each subset is then calculated as:

$$\text{Speed}_{\text{subset } i} = \frac{\text{Length}}{TT_{\text{subset } i}} = \frac{\text{FFS}}{TTI_{\text{subset } i}} \quad (64)$$

- where:
- Speed_{subset i} = average speed for subset *i* (mi/h)
 - Length = length of freeway segment (mi)
 - TT_{subset i} = average travel time for the subset (hr)

Next, the density is calculated for each subset. Since the safety-density relationship is only valid for densities between 20 and 78 pc/mi/ln, calculated densities below or above this range are limited at 20 and 78, respectively.

$$\text{Density}_{\text{subset } i} = (225) \left(1 - \frac{\text{Speed}_{\text{subset } i}}{\text{FFS}} \right) = (225) \left(1 - \frac{1}{\text{TTI}_{\text{subset } i}} \right) \quad (65)$$

where: $\text{Density}_{\text{subset } i}$ = average traffic density for subset i (pc/mi/ln)
 $\text{Speed}_{\text{subset } i}$ = average speed for the subset (mi/h)

Next, the FI and PDO crash rates for each subset are estimated using the safety-density relationship:

$$\begin{aligned} \text{FICR}_{\text{subset } i} = & 1.022 - 0.0842(\text{Density}_{\text{subset } i}) + 0.00264(\text{Density}_{\text{subset } i})^2 \\ & - 0.0000179(\text{Density}_{\text{subset } i})^3 \end{aligned} \quad (66)$$

where: $\text{FICR}_{\text{subset } i}$ = FI crash rate for subset i (crashes/MVMT)

$$\begin{aligned} \text{PDOCR}_{\text{subset } i} = & 1.614 - 0.1301(\text{Density}_{\text{subset } i}) + 0.00444(\text{Density}_{\text{subset } i})^2 \\ & - 0.0000179(\text{Density}_{\text{subset } i})^3 \end{aligned} \quad (67)$$

where: $\text{PDOCR}_{\text{subset } i}$ = PDO crash rate for subset i (crashes/MVMT)

In order to estimate the crash frequencies from the crash rates, the annual travel (MVMT) must be determined. This is calculated as:

$$\text{AMVMT}_{\text{tot}} = \frac{\text{Demand} \times \text{Length} \times N_{\text{days}}}{1,000,000} \quad (68)$$

where: $\text{AMVMT}_{\text{tot}}$ = total annual million vehicles-miles traveled (MVMT/yr)
 Demand = hourly volume for the freeway segment during the hour time-slice (vehicles/hr)
 Length = length of freeway segment (mi)
 N_{days} = number of days in yearly study period (= 250 days as explained in Section 4.2.2)

The annual travel for each subset is then calculated as:

$$\text{AMVMT}_{\text{subset } i} = (\text{AMVMT}_{\text{tot}})(\text{Weight}_{\text{subset } i}) \quad (69)$$

where: $AMVMT_{subset\ i}$ = annual million vehicle-miles traveled for subset I (MVMT/yr)
 $AMVMT_{tot}$ = total annual million vehicle-miles traveled (million vehicle-miles/yr)
 $Weight_{subset\ i}$ = proportion of all the 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 follows:

$$NFI_{tot} = \sum_{i=1}^5 (FICR_{subset\ i})(AMVMT_{subset\ i}) \quad (70)$$

where: NFI_{tot} = total predicted number of fatal-and-injury crashes (crashes/yr)

$$NPDO_{tot} = \sum_{i=1}^5 (PDOCR_{subset\ i})(AMVMT_{subset\ i}) \quad (71)$$

where: $NPDO_{tot}$ = total predicted number of PDO crashes (crashes/yr)

The final values of NPDO and NFI are calculated and recorded first for the untreated TTI curve, based on the five TTI percentiles. Next, this series of calculations is completed using the five percentile values for the treated TTI curve. Using these values, the reductions in FI and PDO crashes can be estimated. This is done using the following equation:

$$\%Reduction_{FI} = \left(1 - \frac{NFI_{tot\ tr}}{NFI_{tot\ unt}} \right) * 100 \quad (72)$$

where: $\%Reduction_{FI}$ = estimated percentage reduction in fatal and major injury crashes due to treatment
 $NFI_{tot\ unt}$ = untreated total predicted number of fatal and major injury crashes (crashes/yr)
 $NFI_{tot\ tr}$ = treated total predicted number of fatal and major injury crashes (crashes/yr)

$$\%Reduction_{PDO} = \left(1 - \frac{NPDO_{tot\ tr}}{NPDO_{tot\ unt}} \right) * 100 \quad (73)$$

where: $\%Reduction_{PDO}$ = estimated percentage reduction in PDO crashes due to treatment
 $NPDO_{tot\ unt}$ = untreated total predicted number of PDO crashes (crashes/yr)
 $NPDO_{tot\ tr}$ = treated total predicted number of PDO crashes (crashes/yr)

Finally, the percent 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:

$$NReduction_{FI} = \left(\frac{\%Reduction_{FI}}{100} \right) (Nexp_{FI}) \quad (74)$$

where: $N\text{Reduction}_{\text{FI}}$ = predicted number of fatal and major injury crashes to be eliminated by treatment (crashes/yr)
 $N\text{exp}_{\text{FI}}$ = number of expected fatal and major injury crashes without treatment (crashes/yr)

$$N\text{Reduction}_{\text{PDO}} = \left(\frac{\% \text{Reduction}_{\text{PDO}}}{100} \right) (N\text{exp}_{\text{PDO}}) \quad (75)$$

where: $N\text{Reduction}_{\text{PDO}}$ = predicted number of PDO crashes to be eliminated by treatment (crashes/yr)
 $N\text{exp}_{\text{PDO}}$ = number of expected PDO crashes without treatment (crashes/yr)

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.

5.4 Estimation of Crash Distributions by Hour of the Day

Chapter 23 of the *Highway Capacity Manual* (2000) (2) provides a methodology for estimating freeway operating speed. In order to determine the operating speed, the free-flow speed (FFS) of the freeway segment is first calculated using the following equation (based on HCM Equation 23-1):

$$\text{FFS} = \text{BFFS} - f_{\text{LW}} - f_{\text{LC}} - f_{\text{N}} - f_{\text{ID}} \quad (76)$$

where: FFS = free-flow speed (mi/h)
 BFFS = base free-flow speed, 70 mi/h (urban) or 75 mi/h (rural)
 f_{LW} = adjustment for lane width from HCM Exhibit 23-4 (mi/h)
 f_{LC} = adjustment for right-shoulder lateral clearance from HCM Exhibit 23-5 (mi/h)
 f_{N} = adjustment for number of lanes from HCM Exhibit 23-6 (mi/h)
 f_{ID} = adjustment for interchange density from HCM Exhibit 23-7 (mi/h)

For $70 < \text{FFS} \leq 75$
 $(3400 - 30\text{FFS}) < v_p \leq 2400$

$$S = \text{FFS} - \left[\left(\text{FFS} - \frac{160}{3} \right) \left(\frac{v_p + 30\text{FFS} - 3400}{30\text{FFS} - 1000} \right)^{2.6} \right] \quad (77)$$

For $55 < \text{FFS} \leq 70$ and for flow rate (v_p)
 $(3400 - 30\text{FFS}) < v_p \leq (1700 + 10\text{FFS})$

$$S = \text{FFS} - \left[\frac{1}{9} (7\text{FFS} - 340) \left(\frac{v_p + 30\text{FFS} - 3400}{40\text{FFS} - 1700} \right)^{2.6} \right] \quad (78)$$

For $55 \leq FFS \leq 75$ and
 $v_p \leq (3400 - 30FFS)$

$$S = FFS \quad (79)$$

where: FFS = free-flow speed (mi/h)
 v_p = 15-min passenger-car equivalent flow rate (pc/h/ln)
 S = operating speed (mi/h)

The average density can then be estimated by dividing the operating speed by the hourly demand volume:

$$\text{Density}_{\text{hour } i} = \frac{S}{\text{Demand}_{\text{hour } i}} \quad (80)$$

where: $\text{Density}_{\text{hour } i}$ = average traffic density for hour i (pc/mi/ln)
 S = operating speed (mi/h)
 $\text{Demand}_{\text{hour } i}$ = 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 using the following equation:

$$\text{CR}_{\text{hour } i} = 2.636 - 0.2143(\text{Density}_{\text{hour } i}) + 0.00708(\text{Density}_{\text{hour } i})^2 - 0.000048(\text{Density}_{\text{hour } i})^3 \quad (81)$$

where: $\text{CR}_{\text{hour } i}$ = total crash rate for hour i (crashes/million vehicle-miles traveled)

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

$$\text{NC}_{\text{hour } i} = \frac{(\text{Demand}_{\text{hour } i})(\text{Length})(250)(\text{CR}_{\text{hour } i})}{1,000,000} \quad (82)$$

where: $\text{NC}_{\text{hour } i}$ = predicted total number of crashes for hour i (crashes/yr)
 Length = length of 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 the following equation:

$$\text{CrashProb}_{\text{hour } i} = \frac{\text{NC}_{\text{hour } i}}{\sum_{i=1}^{24} \text{NC}_{\text{hour } i}} \quad (83)$$

where: $\text{CrashProb}_{\text{hour } i}$ = 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 then describes the calculation procedure to estimate the final benefit-cost ratio. A spreadsheet tool to implement this methodology is described in Section 2.3 .

6.1 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:

- Benefit-cost ratio
- Net present benefits

These measures are defined as:

$$\text{Benefit - Cost Ratio} = B/C \quad (84)$$

$$\text{Net Present Benefits} = B - C \quad (85)$$

where: B = present value of treatment benefits (\$)
C = 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 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 follows:

$$C = IC + AMC (USPWF) \quad (86)$$

where: IC = implementation or construction cost (\$)
 AMC = annual maintenance cost (\$)
 USPWF = uniform series present worth factor

The uniform series present worth factor is defined as:

$$USPWF = \frac{(1 + i)^n - 1}{i (1 + i)^n} \quad (87)$$

where: i = minimum attractive rate of return or discount rate (expressed as a proportion; i.e., i = 0.04 represents a 4 percent discount rate)
 n = service life of design treatment (yr)

The benefits of design treatments in the life-cycle benefit-cost analysis combine both traffic operational and safety benefits:

$$B = (AOB + ASB) USPWF \quad (88)$$

where: AOB = annual traffic operational benefits (\$)
 ASB = annual safety benefits (\$)

Equation (88) is suitable for the current assessment tool which is based on constant traffic volumes. A future potential enhancement of the tool could allow the user to specify an annual percentage growth in traffic volume. Equation (88) would then be replaced by:

$$B = \sum_{j=1}^n (AOB_j + ASB_j) \left(\frac{1}{(1 + i)^j} \right) \quad (89)$$

where: AOB_j = annual traffic operational benefit for year j (\$)
 ASB_j = annual safety benefit for year j (\$)

The term $1/(1 + i)^j$ represents the single-amount present worth factor for year j.

The annual traffic operational benefit for a design treatment is determined as:

$$AOB = \sum_{k=1}^{24} \Delta R_k (VOT) + \Delta \sigma_k (VOR) V_k N_d L \quad (90)$$

where: ΔD_k = change in annual traffic operational delay due to the design treatment during hour k (veh-h)
 VOT = value of travel time (\$/veh-h)
 $\Delta \sigma_k$ = change in the standard deviation of travel time during hour k
 VOR = value of reliability (\$/veh-h)

- V_k = traffic volume on facility during hour k
- N_d = number of days per year (= 250 days)
- 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 future possible enhancement of the tool could incorporate additional operational benefits, such as vehicle operating cost/fuel cost savings and reduced emissions.

The annual safety benefit for a design treatment is determined as:

$$ASB = N\text{Reduction FI} (CC_{FI}) + N\text{Reduction PDO} (CC_{PDO}) + DSB_{FSI} (CC_{FSI}) + DSB_{MI} (CC_{MI}) + DSB_{PDO} (CC_{PDO}) \quad (91)$$

- where:
- 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
 - 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 for which FHWA 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 by the terms $N\text{ReductionFI}$ and $N\text{ReductionPDO}$. The methodology for deriving these terms has been presented in Chapter 5 in Equations (62) through (75).

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

6.1.1 Implementation or Construction Cost (IC)

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

6.1.2 Annual Maintenance Cost (AMC)

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.

6.1.3 Minimum Attractive Rate of Return or Discount Rate (i)

The minimum attractive rate of return or discount rate 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 that they can be compared on a common basis. The suggested default value of the discount rate is 7 percent. This value of the discount rate was chosen based on Office of Management and Budget (OMB) Circular A-94 (8), which specifies a real discount rate of 7 percent for analysis of public investments. Circular A-94 has been U.S. Government policy since 1992, and has been reissued within the last year with the discount rate provision unchanged.

6.1.4 Service Life (n)

The service life of design treatments varies over a broad range from 5 (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.

6.1.5 Change in Annual Traffic Operational Delay (ΔD_k)

The change in annual traffic operational delay for a specific design treatment during a specific hourly time-slice (ΔD_k) is computed with a procedure that is documented in Chapter 4 of this report (also referred to as ΔL_k). ΔD_k is derived directly from the area between the treated and untreated TTI curves using the approximation shown in Figure 4-7 and Equation (47). 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 (47) 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 Section 4.3 of this report.

6.1.6 Change in the Standard Deviation of Travel Time ($\Delta\sigma_k$)

The standard deviation of travel time for a specific design treatment during a specific hourly time-slice ($\Delta\sigma_k$) is computed with a procedure that is documented in Section 4.3.4 of this report. $\Delta\sigma_k$ 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 (49). Then, $\Delta\sigma_k$ is determined as the difference between those standard deviations, as shown here:

$$\Delta\sigma_k = \sigma_{\text{untreated},k} - \sigma_{\text{treated},k} \quad (92)$$

where: $\sigma_{\text{untreated},k}$ = the standard deviation of travel time (h) for the untreated condition, derived from an untreated TTI curve like that shown in Figure 4–4
 $\sigma_{\text{treated},k}$ = the 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

6.2 Values of Travel Time and Reliability (VOT and VOR)

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 (10). The distribution is skewed with a relatively long tail toward higher travel times, as is typical of data for unreliable conditions. The mean travel time shown in the figure 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.

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

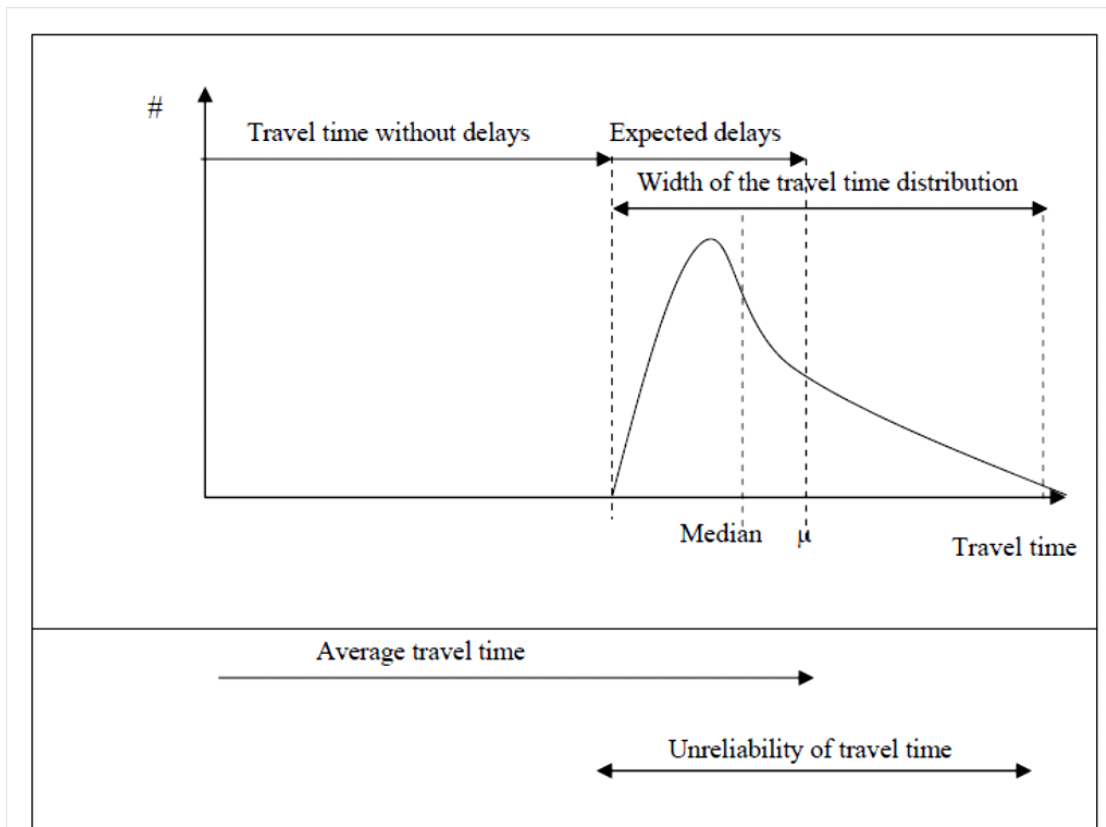


Figure 6–1. Typical Example of Travel Time Distribution Curve Used to Estimate Delay and Reliability (10)

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 that 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.

Since 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, since 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 review of these studies, Concas and Kolpakon (11) made the following

Table 6-1. Results of Studies on the Value of Travel Time and Delay Reduction (11)

Study	Year	Data used	VOT estimate
Becker (12)	1965		40% of wage rate
Beesley (13)	1965	Data from the survey of Government employees in London, UK	31% to 50% of wage rate
Lisco (14)	1967		20% to 51% of wage rate
Miller (15)	1989	Survey of multiple route choice models	60% of gross wage (on average)
Small (16)	1992	Values derived from multiple mode choice transportation models	20% to 100% of gross wage; 50%-reasonable average
Waters (17)	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 (18)	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 (19)	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 (20)	2001	Data on commute travelers on SR-91 in California	Average VOT is \$22.87/hr, or 72% of sample wage rate
Brownstone and Small (21)	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 hr, or 50% to 90% of average wage rate in the sample
USDOT (9)	2003	Estimates are based on multiple sources of data	50% to 120% of the wage rate depending on type of travel (personal vs. business) 50% of wage rate for personal local travel 100% of wage rate for commercial local travel
Small et. al. (22)	2005	Travel from SR-91 in greater Los Angeles area (CA), collected over 10-mo period in 1999 to 2000	Median VOT is \$21.46/hr or 93% of average wage rate
Tseng et. al. (23)	2005	Data for Dutch commuters who drive to work two or more time per week. Collected in June 2004	Mean VOT for all travelers: 10 Euros/hr (approximately \$12.10/hr)
Litman (24)	2007	Results are drawn from multiple travel time studies	25% to 50% of prevailing wage (for personal travel)
Tilahun and Levinson (25)	2007	Data from stated preference survey of travelers on I-394 in Minneapolis/St. Paul area	\$10.62/hr for MnPass (ETC system) subscribers that were early/on-time \$25.42/hr for MnPass subscribers that were late \$13.63/hr for nonsubscribers that were early/on-time \$10.10/hr for non-subscribers that were late

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 percent of the prevailing wage rate
- On-the-clock paid travel (e.g., commercial vehicle driver) should be valued at 100 percent 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 of the national average wage rate from the Bureau of Labor Statistics (26) is \$20.90 per hour for May 2009.

The default value used for the value of travel time in the analysis tool is \$15.68 per hour. Users may replace this value with any value considered more appropriate for their local condition.

6.2.2 Value of Reliability

Warffemius (10) makes the case that the variability (i.e., the variance or standard deviation of the travel-time distribution) is a useful measure of reliability. The greater the variance or 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 follows:

$$\text{VOR} = \rho \text{ VOT} \quad (93)$$

where: ρ = reliability ratio

Warffemius indicates that Copley et al. (27) have estimated the reliability ratio as equal to 1.3 based on a stated preference survey among commuters in Manchester, England, 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 in Section 6.1.6 can be used to implement this concept.

Warffemius further 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 (91).

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 where late arrivals can have significant consequences, while 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 since the arrival time at home is less important than the arrival time at work. The reliability of personal errands or leisurely trips is also expected to be less valuable than a morning commute, since 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 (28). An overview and meta-analysis on this topic completed in 2012, is provided by Carrion and Levinson (29).

Table 6-2. Results of Studies on the Value of Reliability

Study authors	Number of respondents	Trip type	Reliability ratio	
Copley et al. (27)	167	Mostly work commutes	1.3	
Black and Towriss (30)	354	Car travelers	0.79	
Small et al. (31) and (32)	NA	Commute to work	1.3	
Halse and Killi (33)	505	Shippers	0.68	
Parsons Brinckerhoff (34)	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 (30)	NA	Car trips to and from work	0.55	
		All trips in sample	0.70	
Asensio and Matas (35)	NA	NA	0.98	
Noland et al (36)	NA	Commuting	1.27	
Bates (37)	NA	NA	1.1	
Ghosh (38)	NA	NA	1.17	
Yan (39)	NA	NA	1.47	
Small et al. (40)	NA	NA	0.65	
Bhat and Sardesai (41)	NA	NA	0.26	
Hollander (42)	NA	NA	0.10	
Tilahun and Levinson (25)	NA	NA	0.89	
Carrion-Madera and Levinson (29)	NA	NA	0.91	
Hensher (43)	198	Long distance (< 3 h)	0.57	
Small et al. (40)	5,630	Commute	3.5	
Lam and Small (44)	332	Male	0.66	
		Female	1.4	
Small et al. (22)	1,155	Commute	0.91	
Brownstone and Small (21)	601	Commute	0.4	

Since 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.

6.3 Costs of Crashes

6.3.1 Crash Cost Reduction Due to Congestion Reduction ($N_{ReductionFI}$ and $N_{ReductionPDO}$)

The crash cost reduction due to congestion reduction has been estimated based on the crash rate-traffic density relationships presented in Section 2.2 and summarized in Equations (62) through (75).

6.3.2 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.

6.3.3 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 USDOT or the National Safety Council. The benefit-cost analysis methodology developed by the research team uses the default values shown in Table 6-3 below, which were taken from recent USDOT 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 non-incapacitating injury crashes, and \$42,000 for possible injury crashes.

Chapter 7

Analysis Tool and Underlying Equations: Test for Reasonableness

7.1 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.

7.2 Approach

Test scenarios were developed to represent realistic conditions for typical freeway sections; test scenarios for extreme conditions were developed as well. 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, predicting the delay savings and reliability measures for each scenario. Based on the delay savings, safety benefits (direct and indirect), and reliability improvements, the net present benefit of each scenario was calculated. 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 using the manual method.

Table 7-1. Scenarios Tested Using the Manual Method

Scenario	Number of lanes	ADT	Number of incidents	Location
1	2	52,500	500	Orlando
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
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	

Since the manual testing was labor intensive, an automated procedure was developed to re-run the sixteen scenarios described above. Using the automated approach, the results of the sixteen scenarios could be quickly plotted in various ways to identify additional or new unrealistic results that were not identified in the previous iteration.

7.3 Initial Results of Reasonableness Tests

Upon completion of the manual testing, the results 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 to other design treatments. For example, as Figure 7–2 shows, wildlife crash reductions are estimated to provide a very large net present benefit, as compared to the other design treatments. While 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. It is important to note that treatment cost and default values related 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.

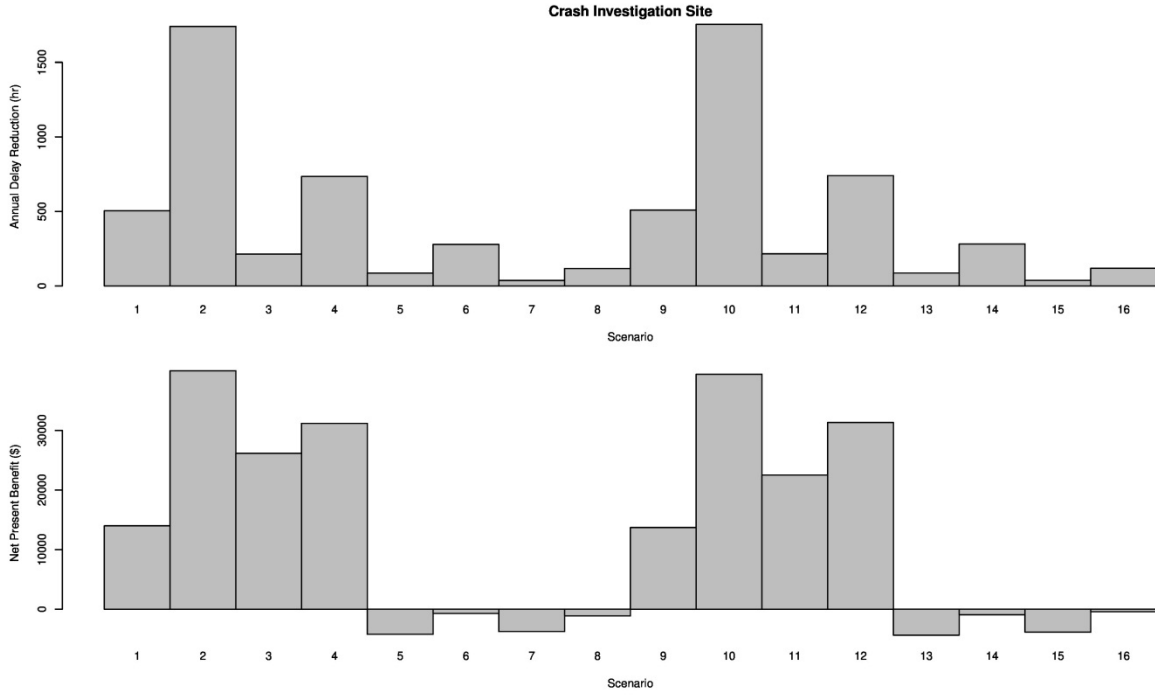


Figure 7-1. Initial Results of Manual Testing—Crash Investigation Site

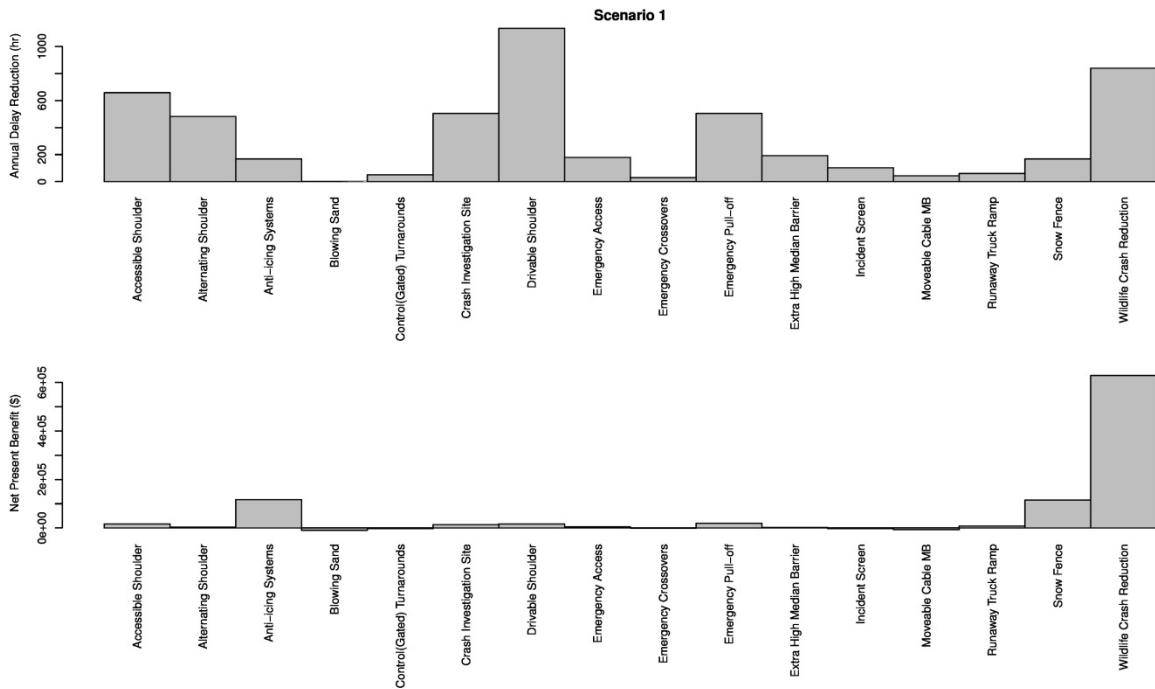


Figure 7-2. Initial Results of Manual Testing—Scenario 1

7.4 Adjustments to Defaults

Based upon 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 percent reduction of PDO, minor-injury crashes, and major-injury crashes (and other non-crash incidents) associated with this design treatment because for most freeway segments, animal-vehicle collisions make up a small proportion of total crashes. Because the model is very sensitive to crash reductions, we chose to err on the side of underestimating benefits with default values.
 - Refined initial values used for default installation cost based on the best available information for wildlife crossing treatments.
 - Set default fatal crash reduction to 0 percent 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 based on the best available information for the installation and maintenance cost of a typical snow fence.
 - 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 based on the best available information on the installation and maintenance costs for such systems.
 - Set default fatal crash reduction to 0 percent 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 percent 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 (NPB) charts shown above.

It should be noted that while 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.

7.5 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. Table 7-2 summarizes the scenarios and codes presented in Figures 7-3 through 7-5.

Table 7-2. Scenarios and Codes Used in Figure 7–3 Through Figure 7–5

Scenario	Location	Number of lanes	Total crash count	Total volume (ADT)	ADT color code	Letter code on top of bars
1	Orlando	2	59	30,000	Light	Number of lanes L or H = low or high crash count
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	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	

7.6 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 makes sense since 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.

Driveable shoulders provide high net-present benefits: Drivable shoulders were found to provide substantial benefits, especially as compared to other design treatments analyzed by the analysis tool (on the typical freeway sections analyzed using the 16 scenarios). Upon 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 negligible. While 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.

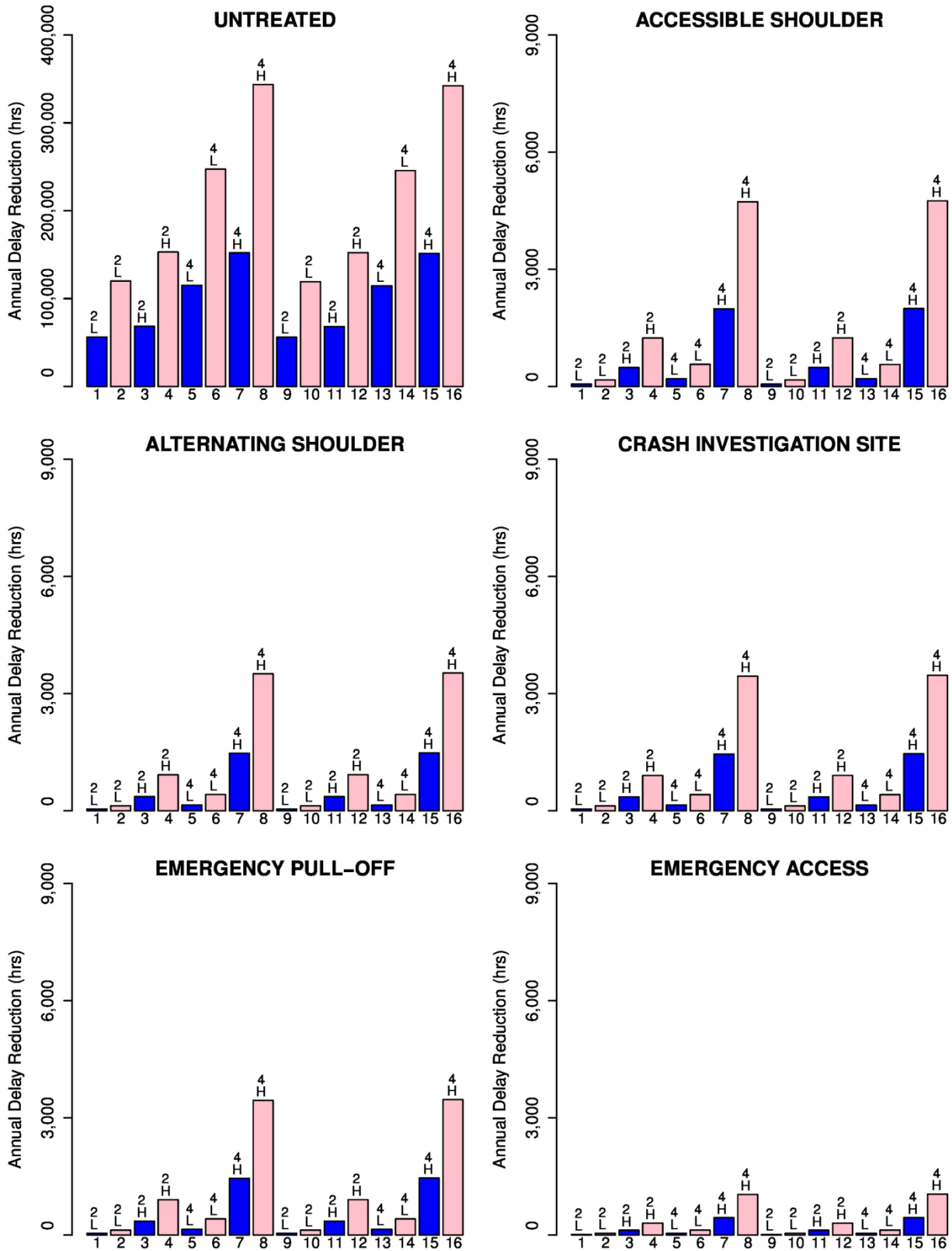


Figure 7-3. Delay Reduction Results

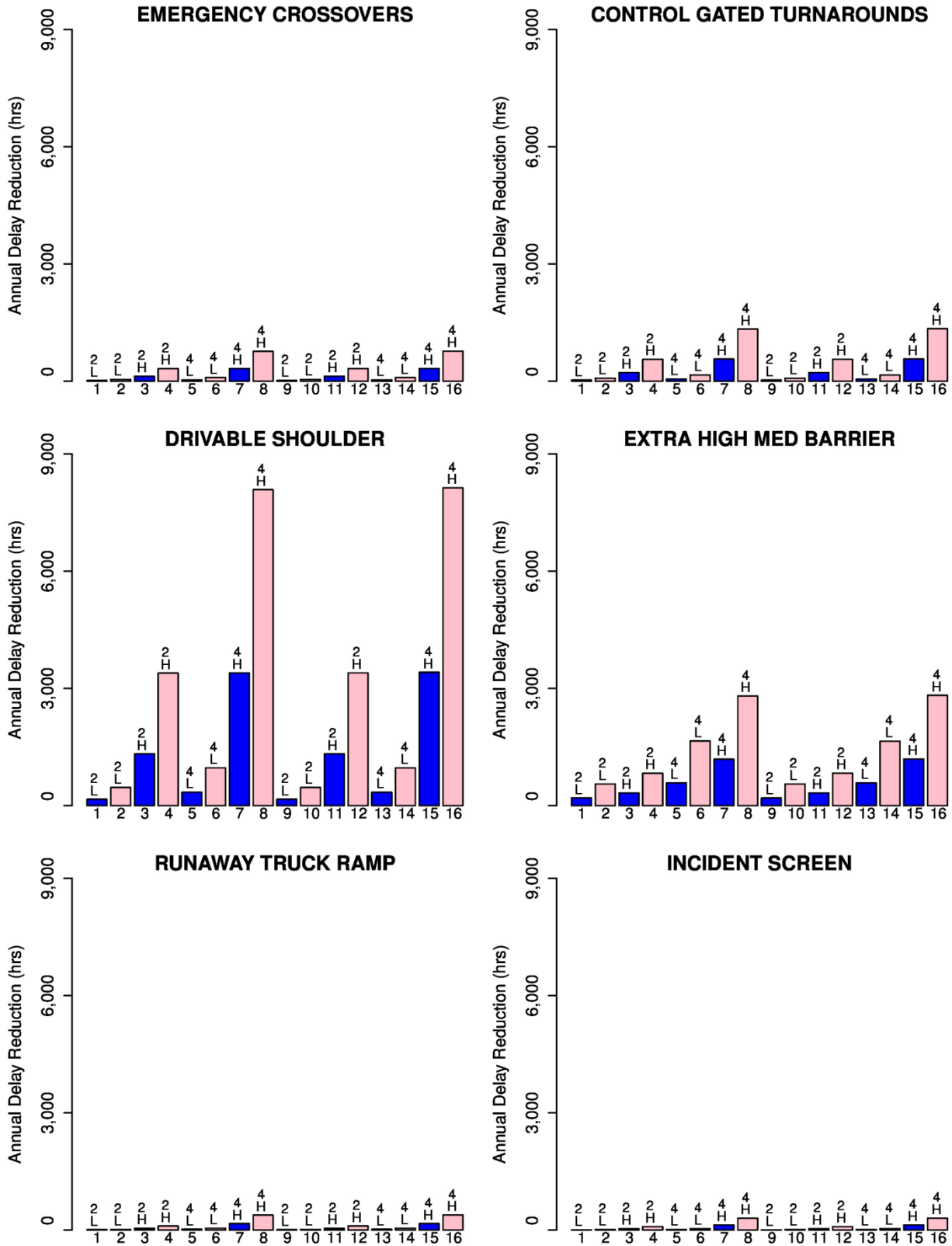


Figure 7-3. Delay Reduction Results (Continued)

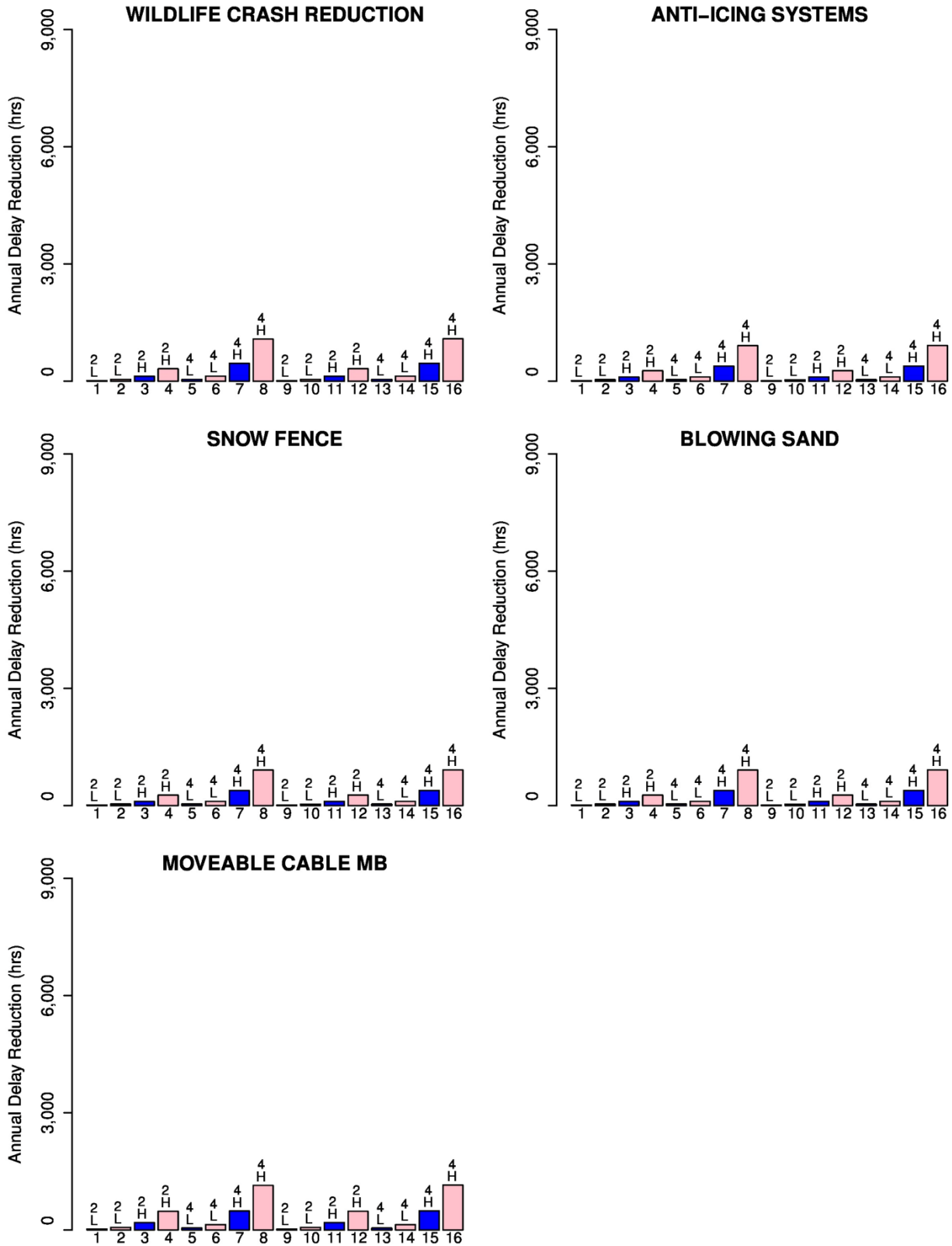


Figure 7-3. Delay Reduction Results (Continued)

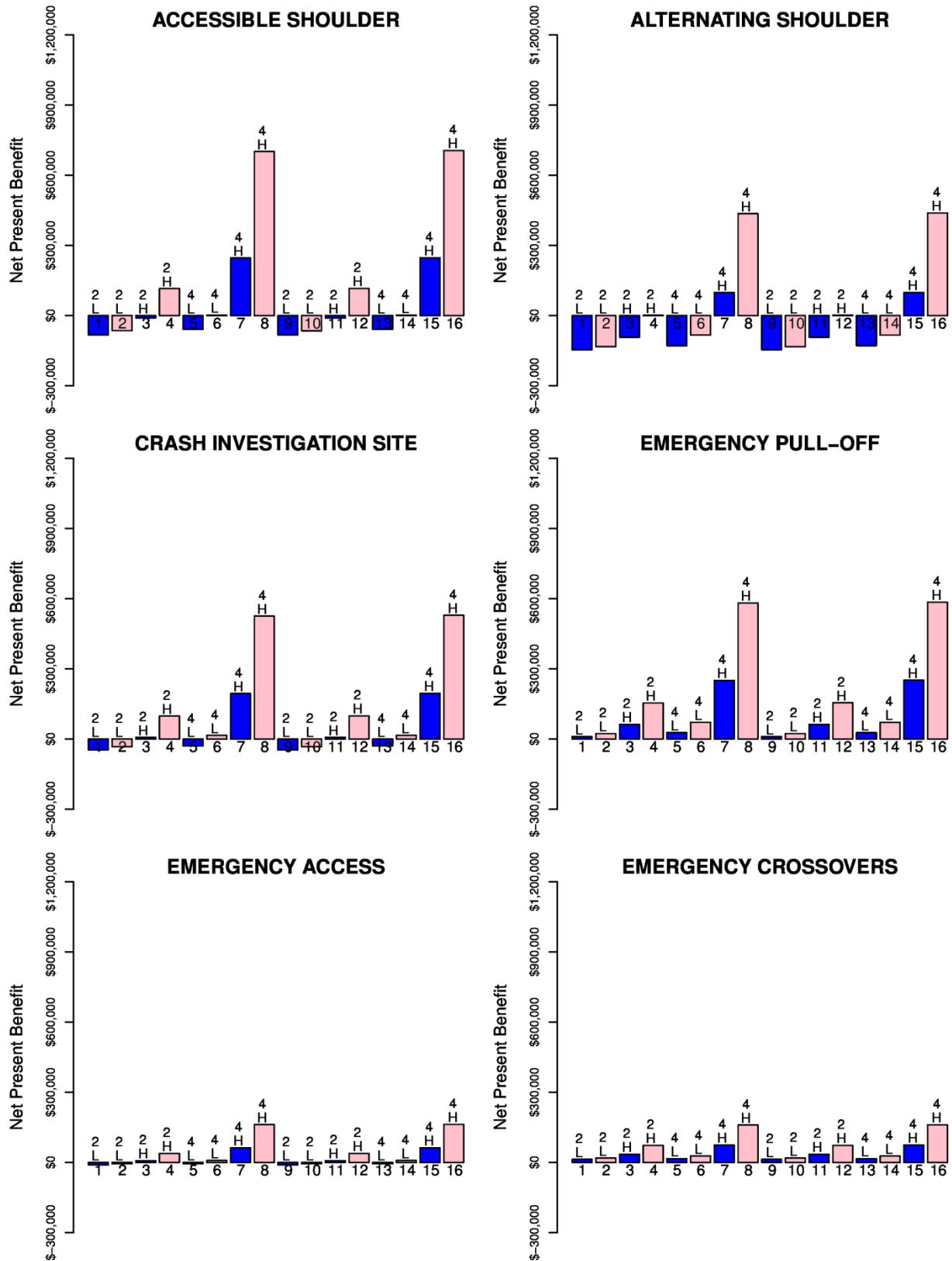


Figure 7-4. Net Present Benefit Results

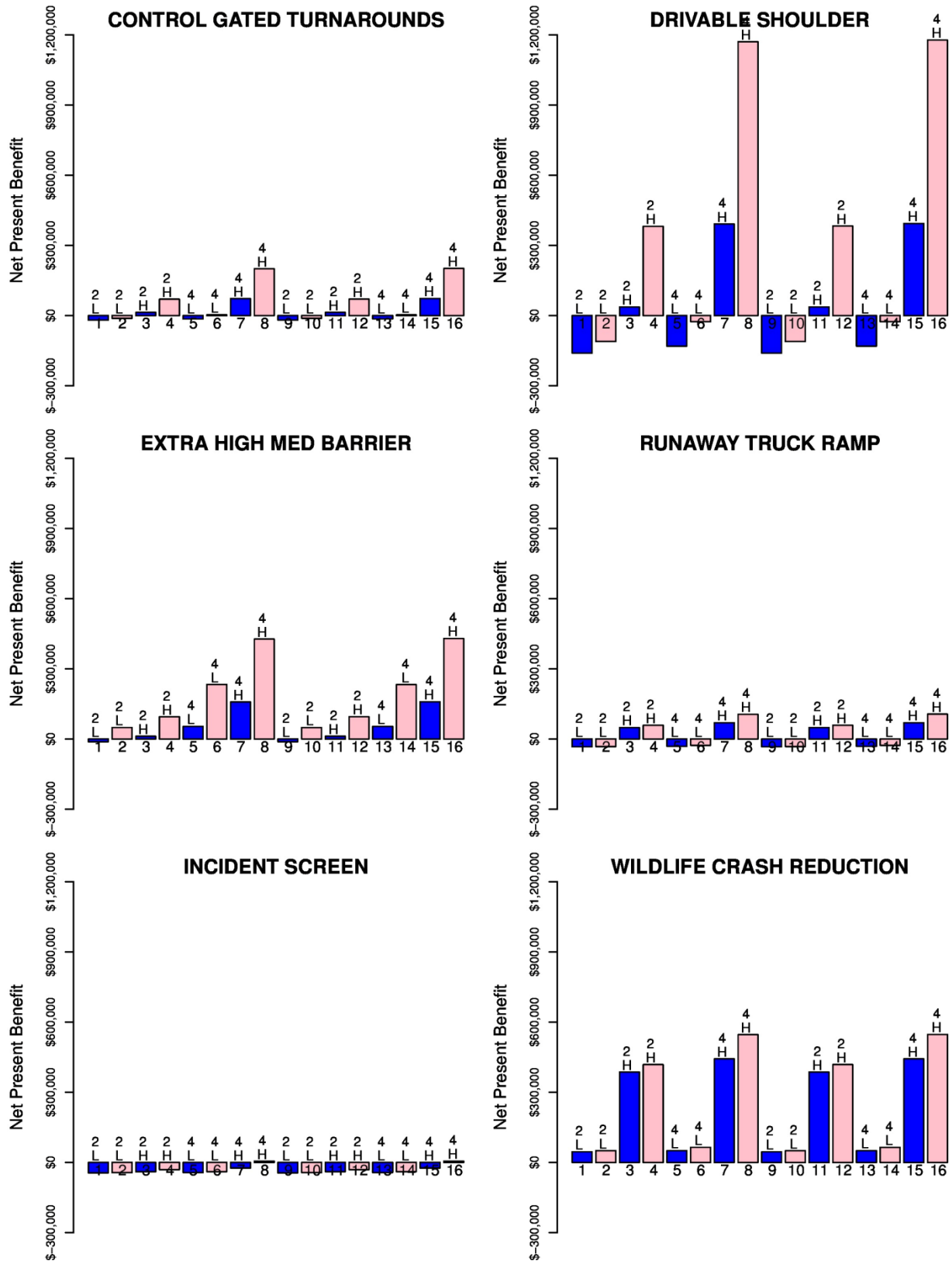


Figure 7-4. Net Present Benefit Results (Continued)

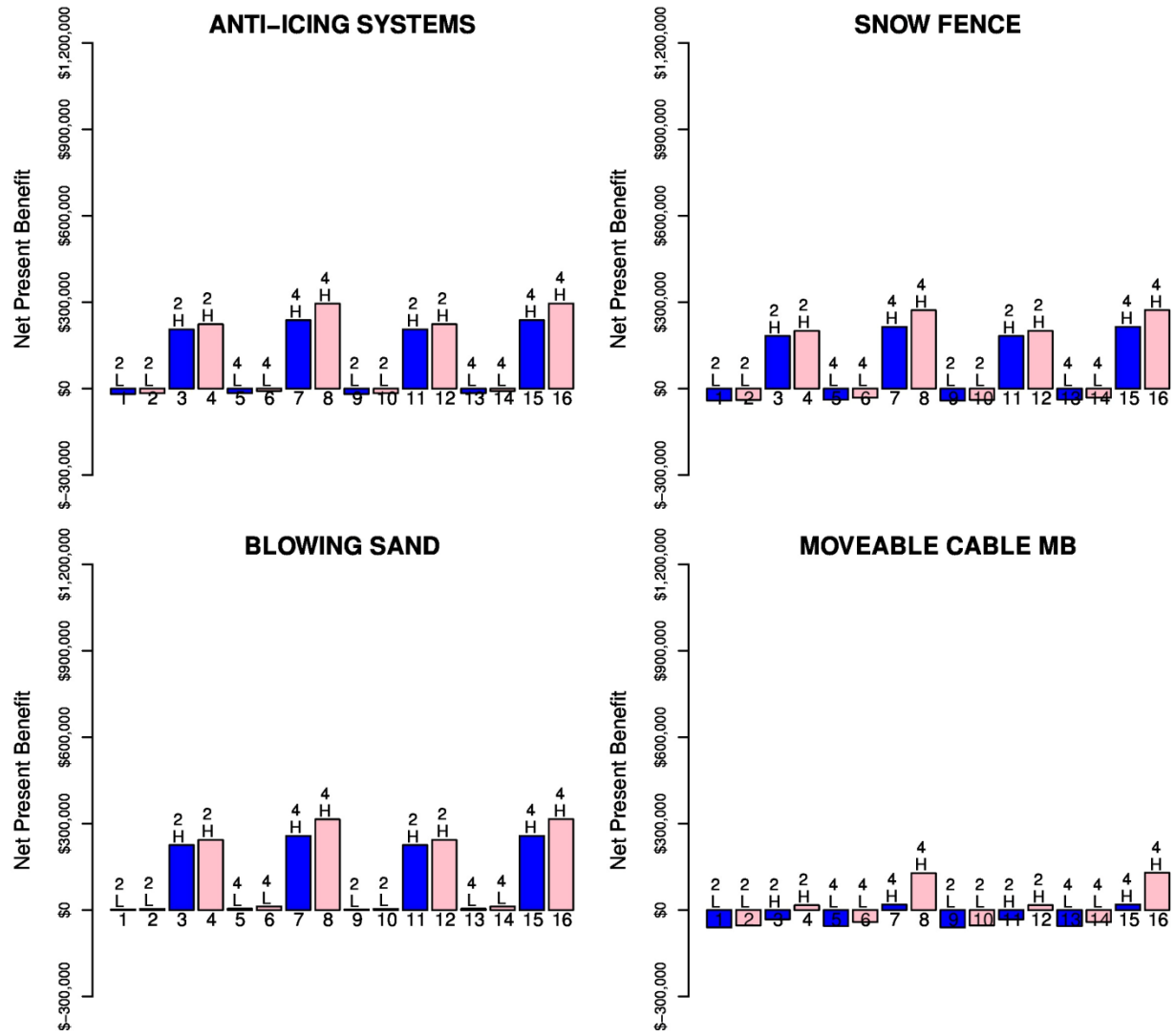


Figure 7-4. Net Present Benefit Results (Continued)

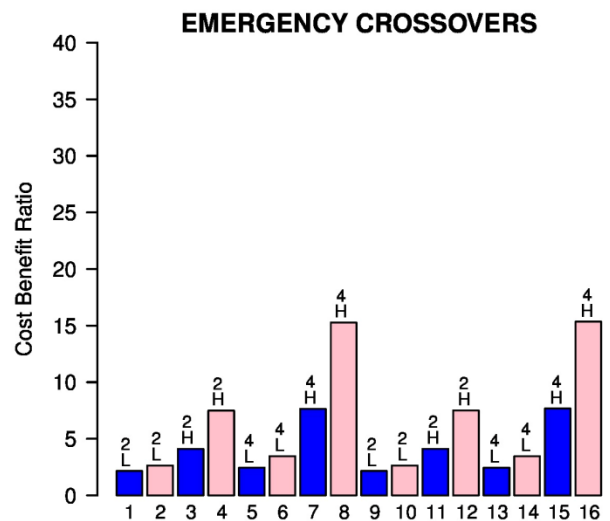
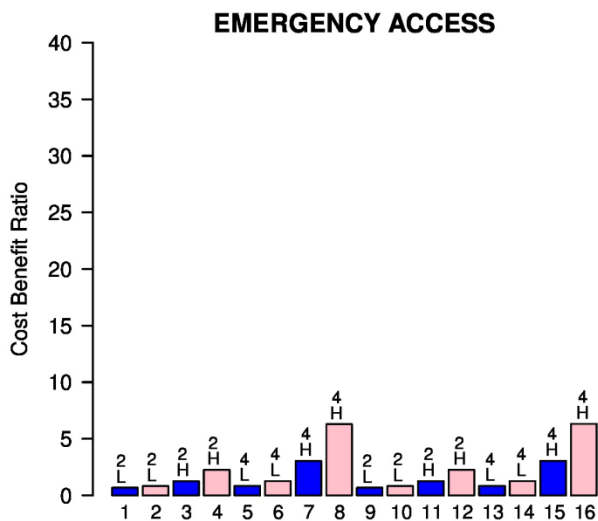
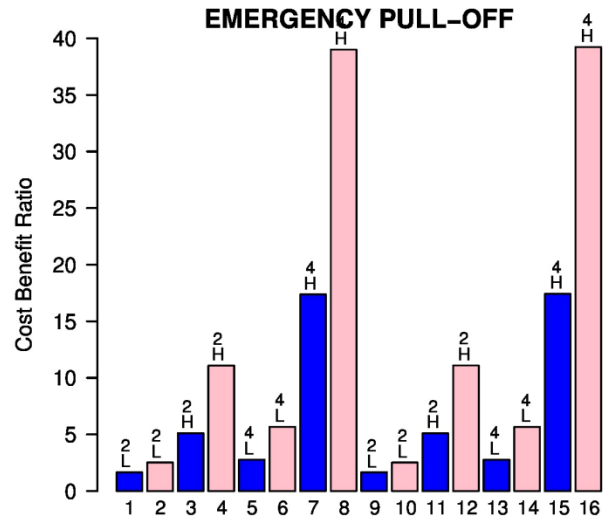
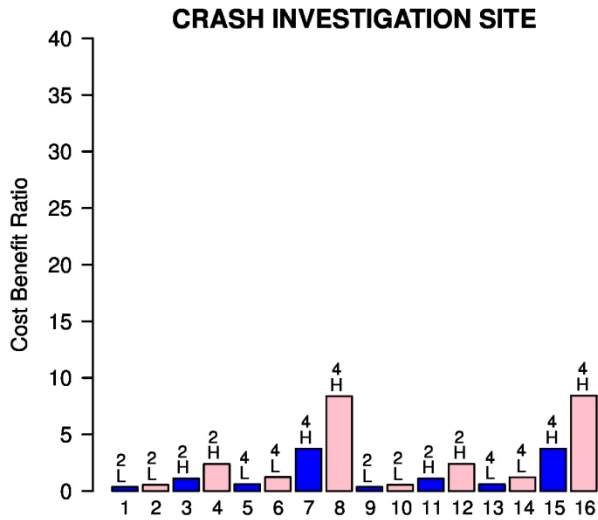
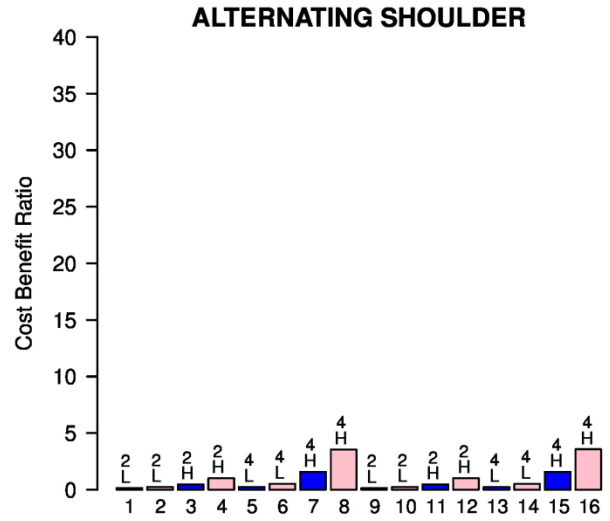
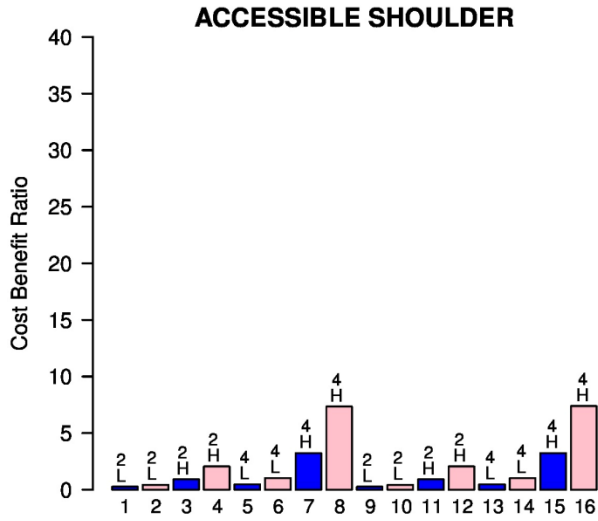


Figure 7-5. Benefit-Cost Ratio Results

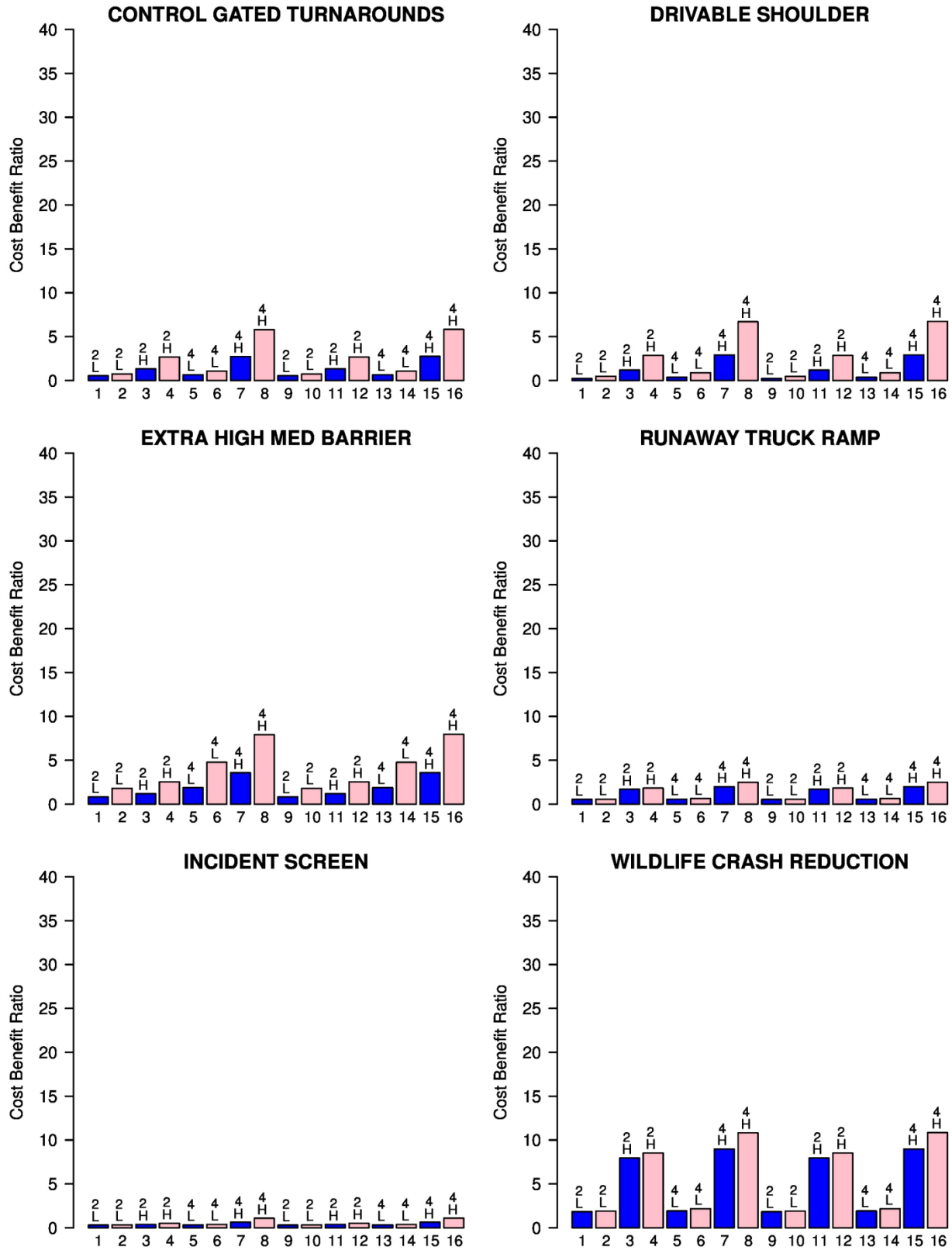


Figure 7-5. Benefit-Cost Ratio Results (Continued)

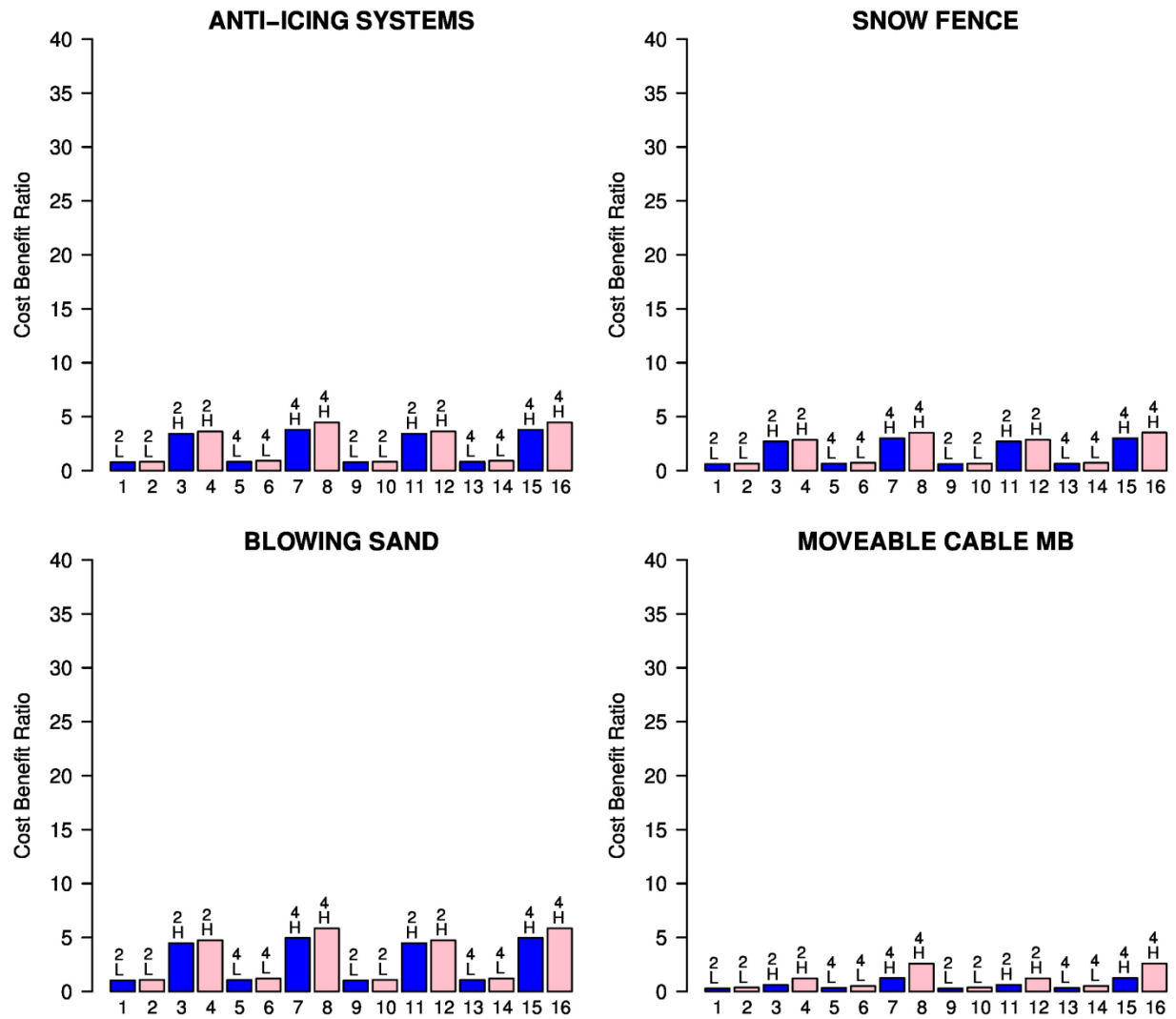


Figure 7-5. Benefit-Cost Ratio Results (Continued)

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 seeking to maximize the potential operational benefits of their freeway design decisions within their resource constraints, including planners, traffic and operational engineers, and managers.

8.1 Conclusions of the Research

8.1.1 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 (ITS) treatments. That is, daily demand peaks that cause rush 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 that 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, and this was frequently done 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 is filling 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.

8.1.2 The 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 percent 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 by previous research in SHRP 2 Project L03. For our purposes, reliability is a measure of the variation in travel times across the segment over a long period of time—we use 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 due to 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 we may find a road to be highly reliable during off-peak hours and not very reliable during peak hours.

Our research shows 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 prior to the start of the event and at the end of 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 nonrecurrent-congestion-causing incidents 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 one 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, while 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.

8.1.3 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 ratio for the roadway segment, all during the particular time-slice 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 upon 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 and/or hours of the day with lower demand-to-capacity ratios. Models were needed that could be applied to all 24 hours 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 using four input variables: rainfall, snowfall, demand-to-capacity 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 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 do 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 incidents and work zones block lanes, 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 have an impact on the demand-to-capacity 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 impact the lane-hours lost or demand-to-capacity ratio input variables is high 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.

8.1.4 The Relationship Between Nonrecurrent Congestion and Safety

This research explored the relationship between congestion and safety—specifically the relationship between Level of Service 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 improvements can be made to the level of service (by implementing design treatments that decrease congestion) in the range from LOS C to LOS F, crash frequency will fall. Therefore, treatments that reduce congestion also improve safety.

8.1.5 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 so 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 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 but received feedback from potential tool users that agencies are capable of easily estimating these costs, and that since construction and materials costs vary so much from location to location as well as over time, any defaults we 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 based on the shift in TTI cumulative curves from untreated to treated. In this project, reliability is 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. We 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 that are 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 this cost to the benefits achieved for recurrent congestion by adding additional capacity.

8.2 Recommendations for Implementation of the Research Results and Future Research Needs

Based on the conclusions of this research effort described above, the 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 have an impact on 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.
- In addition to considering reliability when planning for the design of new facilities or major reconstruction, reliability should be considered when highways are being reconstructed to add capacity for recurrent congestion concerns. While reducing recurrent congestion often also 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 future research needs related to reliability analysis for nonrecurrent congestion and/or potential future enhancements to the tool include:

- 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/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 file and scenario management capabilities of the tool to make analysis of multiple sites easier.
- Expanding the tool to explicitly compare non-design (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 vs. congestion relationship using data from additional cities/regions.

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Appendix A

Background Information for Enhancement of the Project L03 Reliability Models as Part of This Research

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, while these models included a variable, $R_{0.05}$, to account for rainfall, the models 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 d/c. To address these and other issues, SHRP 2 approved an extension of Project L07 to further develop and refine the analytical framework and spreadsheet-based analysis tool that were developed in the research.

This appendix describes in detail the work conducted to:

- Further develop the models to address the effects of snow and ice on the traffic operational effectiveness of design treatments
- Develop reliability models for time periods with $d/c < 0.8$

Further Development of 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 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 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.

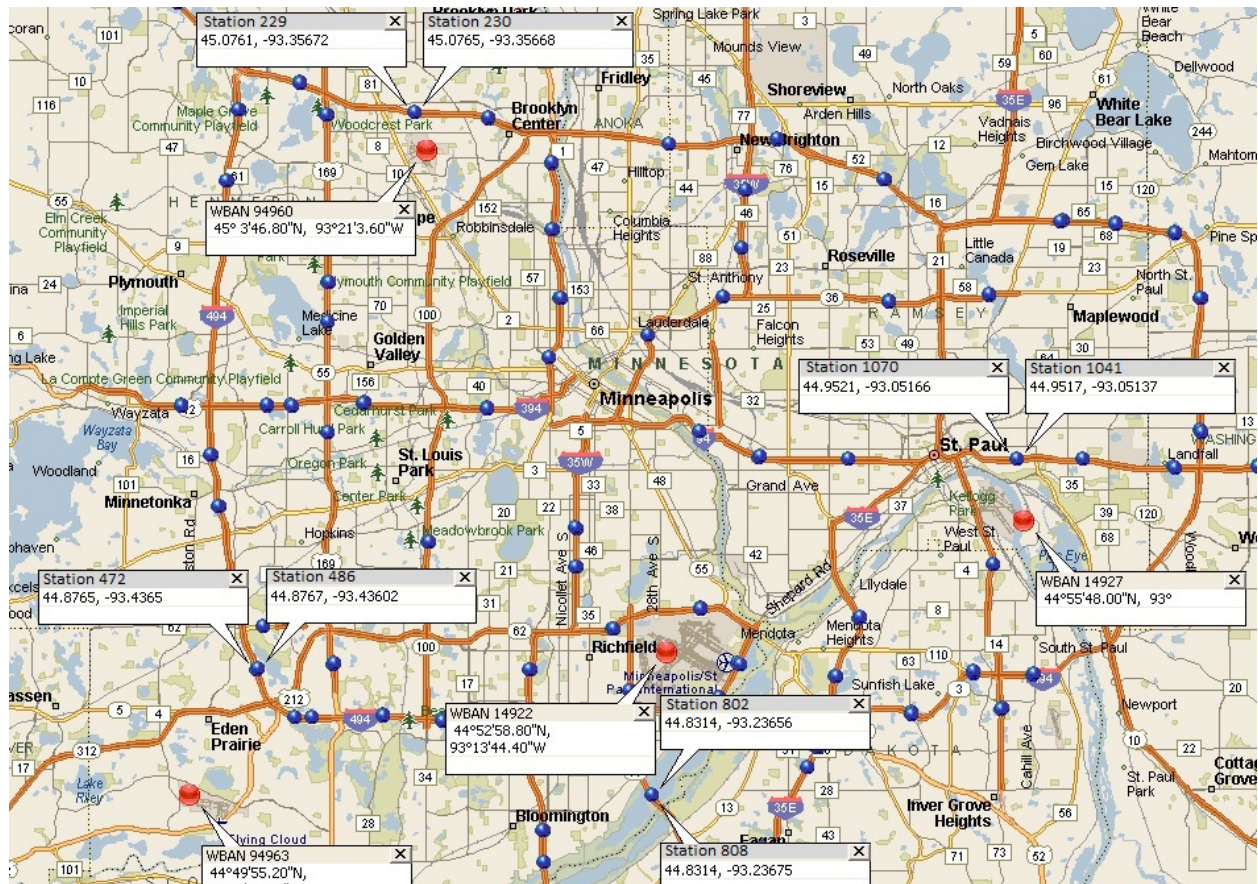


Figure A-1. Freeway Stations Used in Task IV-1 Analysis
 (©Microsoft Streets and Trips)

Speed and volume data for these freeway segments were already available from other analyses in Project L07. Each 5-minute record includes an average per-lane speed and a per-lane volume. In order to determine the minimum snowfall rate which has an effect on travel speeds, the data was filtered in several ways. First, hours with traffic volumes greater than 1,200 veh/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.

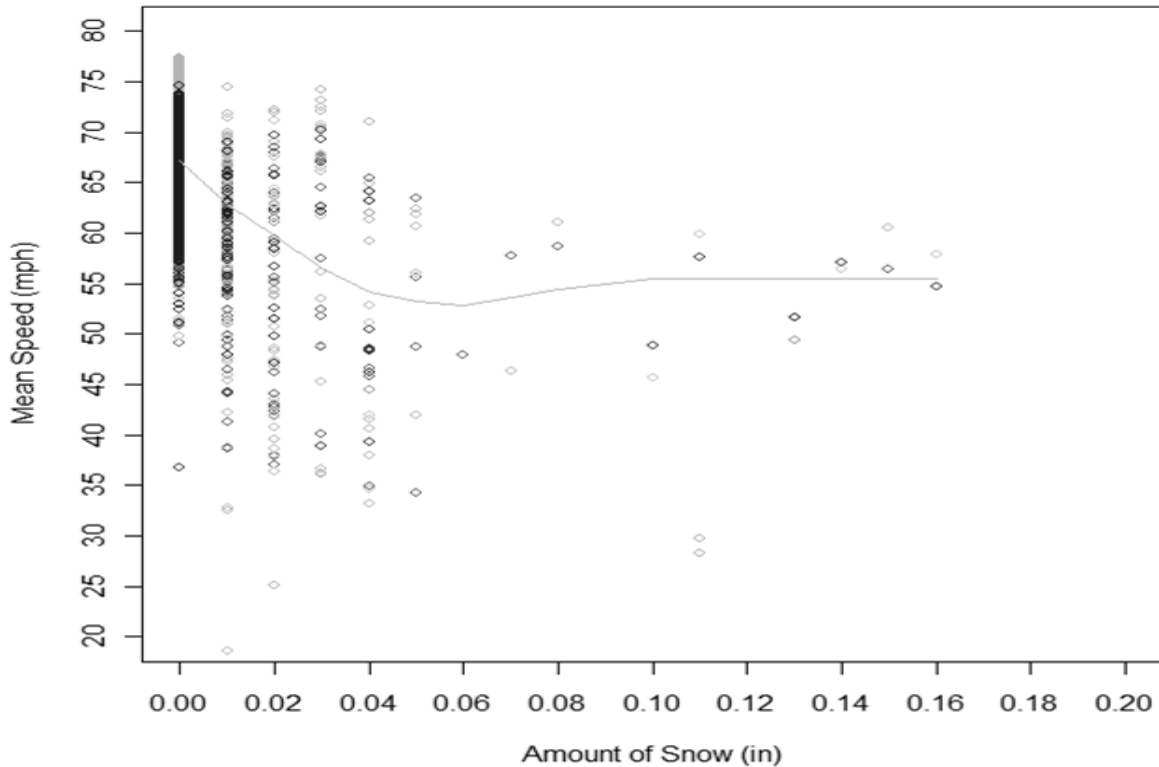


Figure A-2. Mean Hourly Speeds by Hourly Snowfall Amount

Based on this analysis, the research team concluded that the appropriate snow term to be used in the reliability 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 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). 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 (NPTTI_n)

The peak-hour reliability model from Project L03 can be represented as:

$$TTI_{n\%} = e^{(j_n dc_{crit} + k_n LHL + l_n R_{0.05})} \quad (\text{A-1})$$

where: TTI_{n%} = *n*th-percentile TTI
 dc_{crit} = critical demand-capacity 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.)
 j_n, k_n, l_n = coefficients that correspond to the *n*th-percentile TTI (see Table A-1)

Table A-1. Coefficients Corresponding to *n*th-Percentile TTI (No Precipitation)

TTI percentile, n	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.

The 95th-percentile equation is the only percentile from the Project L03 peak-hour model to have a non-zero coefficient for the rain variable. Therefore, the research team used the L03 peak-hour model *without the rain variable* to develop the following TTI for no precipitation:

$$NPTTI_{n\%} = e^{(j_n dc_{crit} + k_n LHL)} \quad (\text{A-2})$$

where: NPTTI_{n%} = *n*th-percentile TTI
 dc_{crit} = critical demand-capacity 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.)
 j_n, k_n = coefficients that correspond to the *n*th-percentile TTI (see Table A-1)

TTI for Hours With Rain Greater Than or Equal to 0.05 in (RTTI_n)

For each of the five TTI percentiles, speed data from those hours with rain ≥ 0.05 in were compared to speed data from those hours with no precipitation, and the following regression equation was developed:

$$RSpeed_n = m * NPSpeed_n + b \quad (\text{A-3})$$

where: RSpeed_n = average *n*th-percentile speed for hours with rainfall ≥ 0.05 in.

$NPSpeed_n$ = average n th-percentile speed for hours with no precipitation;
calculated using the following relationship:

$$NPSpeed_n = \frac{FFS}{NPTTI_n} \quad (A-4)$$

m, b = coefficients for n th-percentile TTI (see Table A-2)

Table A-2. Coefficients Corresponding to n th-Percentile TTI (Rain)

TTI percentile	Type of precipitation	m	b
10	Rain	1.364	-28.34
50	Rain	0.966	-6.74
80	Rain	0.630	6.89
95	Rain	0.639	5.04
99	Rain	0.607	5.27

The variable, $RSpeed_n$, can be converted back into a TTI for rain using the following equation:

$$RTTI_n = \frac{FFS}{RSpeed_n} \quad (A-5)$$

TTI for Hours With Snow Greater Than or Equal to 0.01 in. ($STTI_n$)

For each of the five TTI percentiles, speed data from those hours with snow ≥ 0.01 in. were compared to speed data from those hours with no precipitation, and the following regression equation was developed:

$$SSpeed_n = m * NPSpeed_n + b \quad (A-6)$$

where: $SSpeed_n$ = average n th-percentile speed for hours with snow ≥ 0.01 in
 $NPSpeed_n$ = average n th-percentile speed for hours with no precipitation;
calculated using the following relationship:

$$NPSpeed_n = \frac{FFS}{NPTTI_n} \quad (A-7)$$

m, b = coefficients for n th-percentile TTI (see Table A-3)

Table A-3. Coefficients Corresponding to n th-Percentile TTI (Snow)

TTI percentile	Type of precipitation	m	b
10	Snow	0.178	15.55
50	Snow	0.345	3.27
80	Snow	0.233	5.24
95	Snow	0.286	1.67
99	Snow	0.341	-0.55

The variable, $SSpeed_n$, can be converted back into a TTI for snow using the following equation:

$$STTI_n = \frac{FFS}{SSpeed_n} \quad (A-8)$$

Final Reliability Model Incorporating Rain and Snow

Based on the number of days of each type of precipitation, a weighted average TTI can be calculated as follows:

$$TTI_n = \frac{NPdays * NPPTI_n + Raindays * RTTI_n + Snowdays * STTI_n}{365} \quad (A-9)$$

where: TTI_n = n th-percentile TTI for a 1-hour time-slice over a year
 $NPdays$ = number of days (for 1-hour time-slice) with no precipitation
 $NPPTI_n$ = n th-percentile TTI for days with no precipitation
 $Raindays$ = number of days (for 1-hour time-slice) with rain ≥ 0.05 in
 $RTTI_n$ = n th-percentile TTI for days with rain ≥ 0.05 in
 $Snowdays$ = number of days (for 1-hour time-slice) with snow ≥ 0.01 in
 $STTI_n$ = n th-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 demand-to-capacity ratios (d/c) 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 “as is” 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. This was likely because the subset of data that was 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 less than 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 less than 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 to the predicted cumulative TTI curves (predicted with the L03 reliability

models). In some cases, the observed and predicted curves were very similar; in other cases, they were markedly different.

While the Project L03 research team had developed the reliability models based on roadway sections, 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). Since the input format for the analysis tool (one of the major deliverable 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 data.

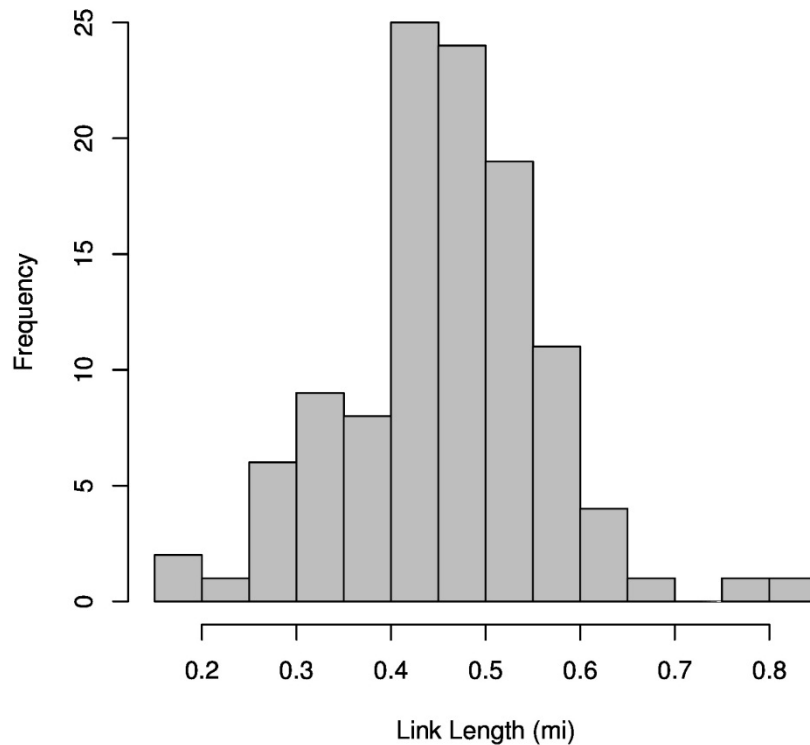


Figure A-3. Histogram of Link Length

Based on the development of the Project L03 models, the time period for predicting a TTI distribution should be one 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 made up one data point in the database, and was abbreviated as HYL. Most of the raw data came in 5-minute, by-lane volumes and speeds, so a procedure was developed to filter and sum the data appropriately. For each hour-year-link (HYL), the following values were determined based on 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 use of the Project L03 models for periods with $d/c < 0.8$ does not produce sufficiently accurate results. We compared the observed values for each percentile to the predicted TTI values 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-5 shows the relative error ($|\text{observed} - \text{predicted}|/\text{observed}$) of the 95th percentile prediction model for the 2006 Minneapolis data. Many of the points are above a relative error of 0.3, meaning that the prediction is off by over 30 percent.

Figure A-5 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 as well. Together, these graphs show that TTI distributions for HYL combinations with very low d/c values tend to vary widely. This 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 which blocks several lanes. The research team has 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

In order 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 had been 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. The variable $S_{0.01}$ is defined as the number of hours per year during a particular time-slice when snowfall exceeds 0.01 inches.

d/c

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.

LHL

LHL represents the sum of the time where a lane (or shoulder) was blocked by a crash-involved or disabled vehicle, or by a work zone. The raw data from Minneapolis included records from a Traffic Management Center (TMC) 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.

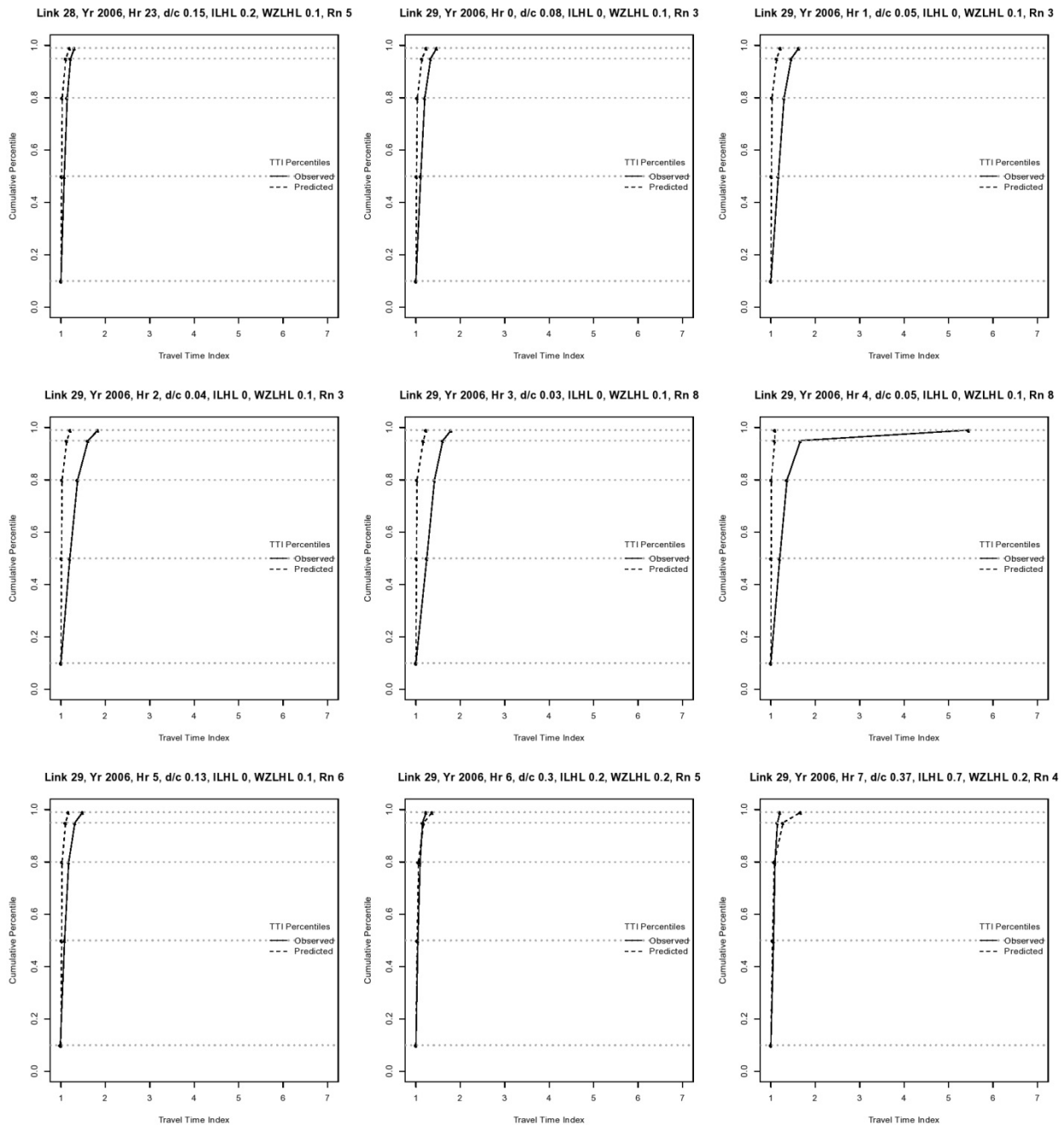


Figure A-4. Excerpt of Figures Comparing Observed TTI Distributions to L03 Predictions

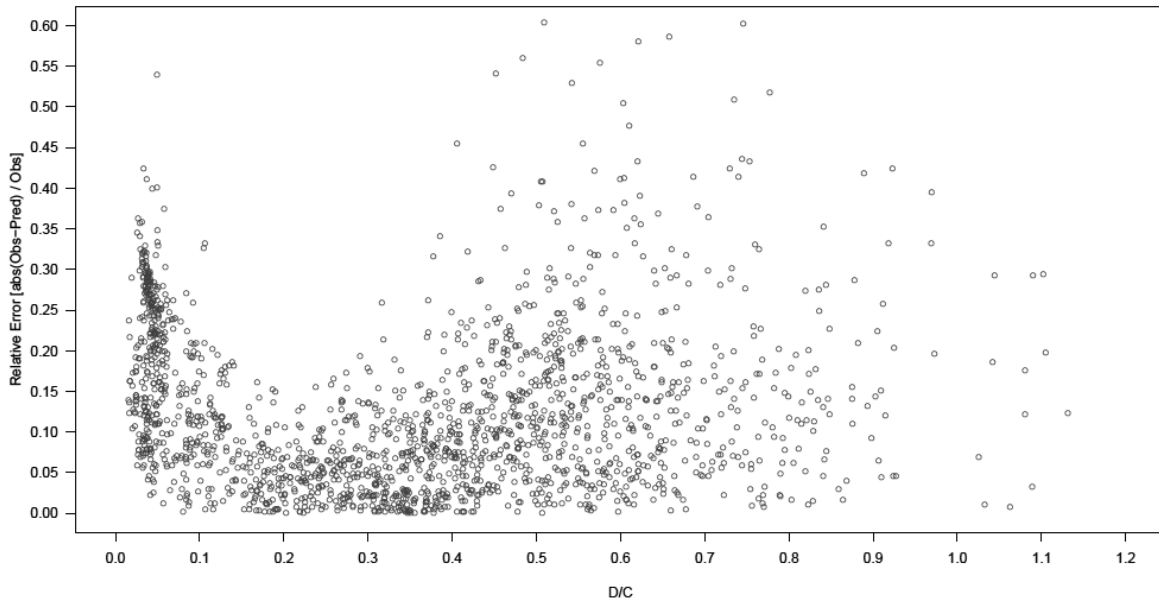


Figure A-5. Relative Error of L03 95th Percentile TTI Models

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. Conversely, 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.

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

Construction event	Number of links blocked by construction event	Total number of links in the section	Percent of 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

As shown in Table A-4, the percentage of a roadway section that was blocked by construction events varied from 14 to 100 percent. 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 percent. 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 ILHL and WZLHL) 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 pm). The bars are colored by LHL cause to show the types of events which contribute most to the total LHL value.

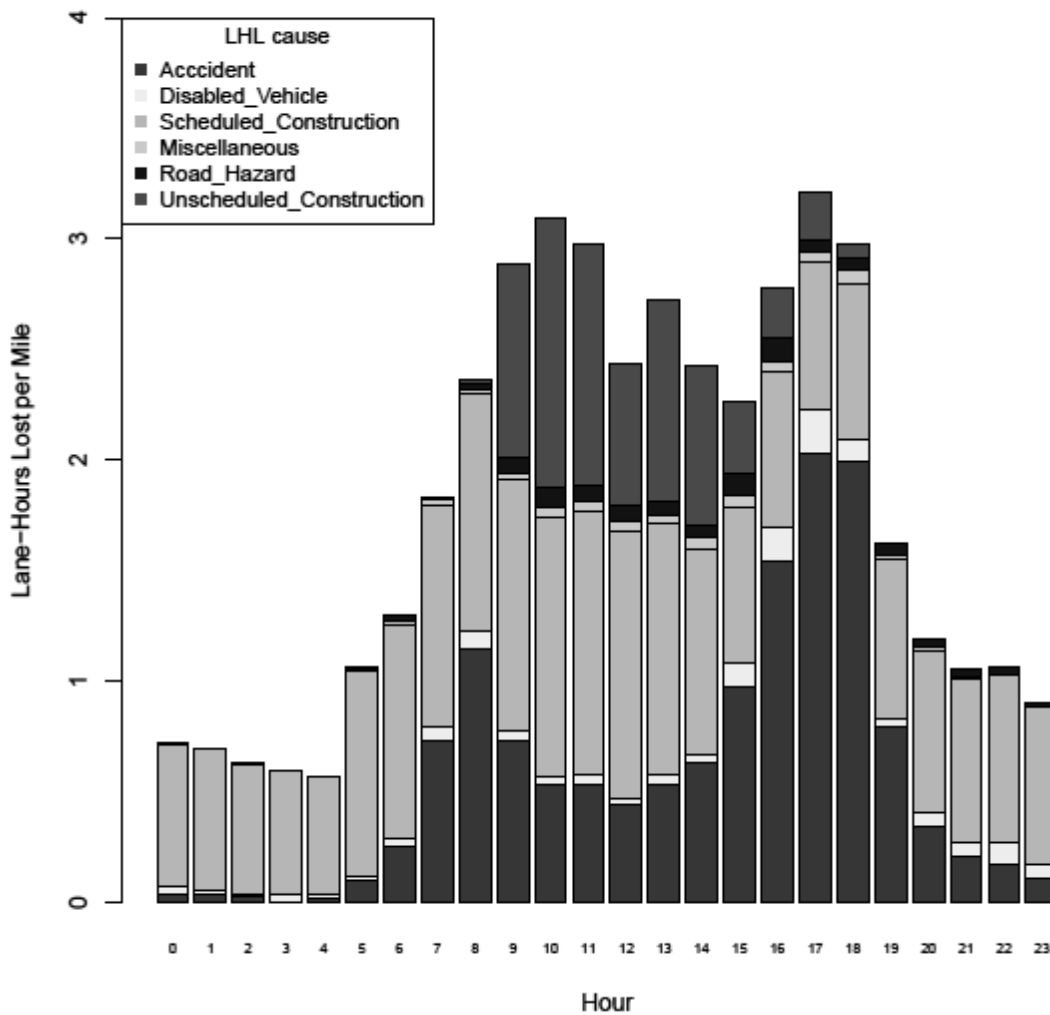


Figure A-6. Average LHL per Mile by Hour

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

$R_{0.05}$ is the number of times (during a given hour over the course of a year) where rainfall exceeds 0.05 inches. $S_{0.01}$ is the number of times snowfall exceeds 0.01 inches. 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. 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 was 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 area ranged from two to ten and $S_{0.01}$ values ranged from 0 to 6.

Final Models

The final database included 1,810 records. Each record represented a single HYL and included values for d/c , $ILHL$, $WZLHL$, $R_{0.05}$, 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 follows:

$$TTI_i = e^{\left(a_i \times \frac{d}{c}\right) + (b_i \times LHL) + (c_i \times R_{0.05}) + (d_i \times S_{0.01})} \quad (\text{A-10})$$

where: TTI_i = cumulative Travel Time Index (TTI) at percentile i
 d/c = demand/capacity ratio
 LHL = lane hours lost
 $R_{0.05}$ = number of hours with rain greater than or equal to 0.05 in
 $S_{0.01}$ = number of hours with snow greater than or equal to 0.01 in
 a_i, b_i, c_i, d_i = coefficients at percentile i

The coefficients “a,” “b,” “c,” and “d” are calculated for a given percentile using the following equation:

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

where: i = a given percentile value (between 0 and 1)
 w, x, y, z = 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