CHAPTER 25
FREEWAY FACILITIES: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

CHAPTER SCOPE

Chapter 25 is the supplemental chapter for Chapter 10, which describes the core methodology for freeway facilities, and Chapter 11, which presents a methodology for evaluating freeway reliability and active traffic and demand management (ATDM) strategies. The computations used by these methodologies are detailed in this supplemental chapter. The documentation is closely tied to FREEVAL-2015E, the computational engine for Chapter 10 and Chapter 11.

The FREEVAL (FREeway EVALuation) tool was initially developed for the 2000 edition of the *Highway Capacity Manual* (HCM) (*1, 2*) and has been updated to reflect subsequent methodological changes in the HCM. All variable definitions and subroutine labels presented in this chapter are consistent with the computational code in FREEVAL-2015E. The Technical Reference Library in Volume 4 contains a FREEVAL-2015E user guide, which provides more details on how to use the computational engine. Other software implementations of this method are available and can be used instead of the computational engine.

CHAPTER ORGANIZATION

Section 2 presents a glossary of all relevant variables used in the procedures and the computational engine. Section 3 and Section 4, respectively, provide details of the undersaturated and oversaturated flow procedures. Section 5 describes details for work zone analysis. Section 6 develops the planning-level methodology for freeway facilities, and Section 7 discusses the mixed-flow model for composite grades. Section 8 develops the freeway calibration methodology at three levels. Section 9 discusses freeway scenario generation, and Section 10 presents an overview of the computational engine structure. Example problems are presented in Section 11, and Section 12 provides references for the chapter.

LIMITATIONS OF THE METHODOLOGIES

The completeness of the analysis will be limited if freeway segment cells in the first time interval, the final time interval, and the first freeway segment do not have demand-to-capacity ratios of 1.00 or less. The methodology can handle congestion in the first interval properly, although it will not quantify any congestion that could have occurred before the first time interval. To ensure a complete quantification of the effects of congestion, it is recommended that the analysis contain an initial undersaturated time interval. If all freeway segments in the final time interval do not exhibit demand-to-capacity ratios less than 1.00, congestion will continue beyond the final time interval, and additional time intervals should be added. This fact will be noted as a difference between the vehicle miles of travel desired at the end of the analysis (demand flow) and the corresponding vehicle miles of travel flow generated (volume served). If queues extend upstream of the first segment, the analysis will not account for the congestion outside the freeway facility but will store the vehicles vertically until the congestion clears the first segment. The same process is followed for queues on on-ramp segments.

The methodology for oversaturated conditions described in this chapter is based on concepts of traffic flow theory and assumes a linear speed–flow relationship for densities greater than 45 passenger cars per mile per lane (pc/mi/ln). This relationship has not been extensively calibrated for field observations on U.S. freeways, and analysts should therefore perform their own validation from local data to obtain additional confidence in the results of this procedure. For an example of a validation exercise for this methodology, the reader is referred elsewhere (*3*).

The procedure described here becomes extremely complex when the queue from a downstream bottleneck extends into an upstream bottleneck, causing a queue interaction. When such cases arise, the reliability of the methodology is questionable, and the user is cautioned about the validity of the results. For heavily congested directional freeway facilities with interacting bottleneck queues, a traffic simulation model might be more applicable. Noninteracting bottlenecks are addressed by the methodology.

The procedure focuses on analyzing a directional series of freeway segments. It describes the performance of a facility but falls short of addressing the broader transportation network. The analyst is cautioned that severe congestion on a freeway—especially freeway on-ramps—is likely to affect the adjacent surface street network. Similarly, the procedure is limited in its ability to predict the impacts of an oversaturated off-ramp and the associated queues that may spill back onto the freeway. Alternative tools are suitable to evaluate these impacts.

2. GLOSSARY OF VARIABLE DEFINITIONS

OVERVIEW

This glossary defines internal variables used exclusively in the freeway facilities methodology. The variables are consistent with those used in the computational engine for the freeway facilities methodology.

If a managed lane facility is adjacent to the general purpose lanes, the oversaturated freeway facilities methodology will analyze each facility independently. As a result, the variables presented in this chapter will pertain to general purpose and managed lane facilities separately.

The glossary of variables is presented in seven parts: global variables, segment variables, node variables, on-ramp variables, off-ramp variables, facilitywide variables, and travel time reliability variables. Global variables are used across multiple aspects of the procedure. Segment variables represent conditions on segments. Node variables denote flows across a node connecting two segments. On- and off-ramp variables correspond to flow on ramps. Facilitywide variables pertain to aggregate traffic performance over the entire general purpose or managed lane facility. Reliability variables pertain to traffic performance over a period of up to one year.

In addition to the spatial categories listed above, there are temporal divisions that represent characteristics over a time step or a time interval. The first dimension associated with each variable specifies whether the variable refers to segment or node characteristics. The labeling scheme for nodes and segments is such that segment i is immediately downstream of node i. The distinction of nodes and segments is used primarily in the oversaturated flow regime as discussed in Section 4.

Thus, there is always one more node than the number of segments on a facility. The second and third dimensions denote a time step t and a time interval p. Facility variables are estimates of the average performance over the length of the facility. The units of flow are in vehicles per time step. The selection of the time step size is discussed later in this chapter.

The variable symbols used internally by the computational engine and replicated in this chapter frequently differ from the symbols used elsewhere in the HCM, particularly in Chapter 10, Freeway Facilities Core Methodology. For example, the HCM uses n to represent the number of segments forming a facility, whereas the computational engine and this chapter use NS.

GLOBAL VARIABLES

- i—index to segment or node number: $i = 1, 2, \ldots, NS$ (for segments) and $i = 1, 2, \ldots, NS + 1$ (for nodes). In the computational engine, i is represented as the index of the *GPSegments/MLSegments Array List* variable in the Seed class.
- KC—ideal density at capacity in vehicles per mile per lane (veh/mi/ln). The density at capacity is 45 pc/mi/ln, which must be converted to vehicles per mile per lane by using the heavy-vehicle adjustment factor

f_{HV} described in Chapter 12, Basic Freeway and Multilane Highway Segments.

- *KJ*—facilitywide jam density (veh/mi/ln).
- *NS*—number of segments on the facility. *NS* is represented as the size of the *GPSegments/MLSegments ArrayList* variable in the Seed class.
- *P*—number of (15-min) analysis periods in the study period. Represented as *Period* in the computational engine. For a 24-h analysis, the theoretical maximum is 96 analysis periods.
- *p*—analysis period index: $p = 1, 2, \ldots, P$.
- *S*—number of computational time steps in an analysis period (integer). *S* is represented as *Step* in the computational engine. *S* is set as a constant of 60 in the computational engine, corresponding to a 15-s interval and allowing a minimum segment length of 300 ft.
- *t*—time step index in a single analysis period: $t = 1, 2, \ldots, S$.
- *T*—number of time steps in 1 h (integer). *T* is set as a constant of 240 in the computational engine, or equal to four times the value of *S*.
- α—fraction of capacity drop in queue discharge conditions due to congestion on the facility. This variable is represented as *inCapacityDropPercentage* in the GPMLSegment class in the computational engine.

SEGMENT VARIABLES

- $ED(i, p)$—expected demand (veh/h) that would arrive at segment *i* on the basis of upstream conditions over time interval *p*. The upstream queuing effects include the metering of traffic from an upstream queue but not the spillback of vehicles from a downstream queue.
- $K(i, p)$—average traffic density (veh/mi/ln) of segment *i* over time interval *p* as estimated by the oversaturated procedure. This variable is represented as the *scenAllDensity_veh* variable in the GPMLSegment class in the computational engine.
- $KB(i, p)$—background density: segment *i* density (veh/mi/ln) over time interval *p* assuming there is no queuing on the segment. This density is calculated by using the expected demand on the segment in the corresponding undersaturated procedure in Chapters 12, 13, and 14.
- $KQ(i, t, p)$—queue density: vehicle density (veh/mi/ln) in the queue on segment *i* during time step *t* in time interval *p*. Queue density is calculated on the basis of a linear density–flow relationship in the congested regime.
- $L(i)$—length of segment *i* (mi). This variable converts the *inSegLength_ft* variable (in feet) to miles when necessary in equations.
- $N(i, p)$—number of lanes on segment *i* in time interval *p*. It could vary by time interval if a temporary lane closure is in effect. *N* is represented as

the *inMainlineNumLanes* variable in the GPMLSegment class in the computational engine.

- *NV*(*i*, *t*, *p*)—number of vehicles present on segment *i* at the end of time step *t* during time interval *p*. The number of vehicles is initially based on the calculations of Chapters 12, 13, and 14, but, as queues grow and dissipate, input–output analysis updates these values in each time step.
- *Q*(*i*, *t*, *p*)—total queue length on segment *i* at the end of time step *t* in time interval *p* (ft).
- *SC*(*i*, *p*)—segment capacity: maximum number of vehicles (veh/h) that can pass through segment *i* in time interval *p* based strictly on traffic and geometric properties. These capacities are calculated by using Chapters 12, 13, and 14. Segment capacity is represented as the *scenMainlineCapacity_veh* variable in the GPMLSegment class in the computational engine.
- *SD*(*i*, *p*)—segment demand: desired flow rate (veh/h) through segment *i* including on- and off-ramp demands in time interval *p* (veh). This segment demand is calculated without any capacity constraints. It is represented as the *scenMainlineDemand_veh* variable in the GPMLSegment class in the computational engine.
- *SF*(*i*, *t*, *p*)—segment flow (veh/h) out of segment *i* during time step *t* in time interval *p* (veh).
- *U*(*i*, *p*)—average space mean speed over the length of segment *i* during time interval *p* (mi/h). It is represented as the *scenSpeed* variable in the GPMLSegment class in the computational engine.
- *UV*(*i*, *t*, *p*)—unserved vehicles: the additional number of vehicles stored on segment *i* at the end of time step *t* in time interval *p* due to a downstream bottleneck.
- *WS*(*i*, *p*)—wave speed: speed at which a front-clearing queue shock wave travels through segment *i* during time interval *p* (ft/s).
- *WTT*(*i*, *p*)—wave travel time: time taken by the shock wave traveling at wave speed *WS* to travel from the downstream end of segment *i* to the upstream end of the segment during time interval *p*, in time steps.

NODE VARIABLES

- *MF*(*i*, *t*, *p*)—actual mainline flow rate that can cross node *i* during time step *t* in time interval *p*.
- *MI*(*i*, *t*, *p*)—maximum mainline input: maximum flow desiring to enter node *i* during time step *t* in time interval *p*, based on flows from all upstream segments and taking into account all geometric and traffic constraints upstream of the node, including queues accumulated from previous time intervals.
- *MO*1(*i*, *t*, *p*)—maximum Mainline Output 1: maximum allowable mainline flow rate across node *i* during time step *t* in time interval *p*, limited by the flow from an on-ramp at node *i*.

- *MO*2(*i*, *t*, *p*)—maximum Mainline Output 2: maximum allowable mainline flow rate across node *i* during time step *t* in time interval *p*, limited by available storage on segment *i* due to a downstream queue.
- *MO*3(*i*, *t*, *p*)—maximum Mainline Output 3: maximum allowable mainline flow rate across node *i* during time step *t* in time interval *p*, limited by the presence of queued vehicles at the upstream end of segment *i* while the queue clears from the downstream end of segment *i*.

ON-RAMP VARIABLES

- *ONRC*(*i*, *p*)—geometric carrying capacity of on-ramp at node *i* during time interval *p*.
- *ONRD*(*i*, *p*)—demand flow rate for on-ramp at node *i* in time interval *p*.
- *ONRF*(*i*, *t*, *p*)—actual ramp flow rate that can cross on-ramp node *i* during time step *t* in time interval *p*; it takes into account control constraints (e.g., ramp meters).
- *ONRI*(*i*, *t*, *p*)—input flow rate desiring to enter the merge point at on-ramp *i* during time step *t* in time interval *p*, based on current ramp demand and ramp queues accumulated from previous time intervals.
- *ONRO*(*i*, *t*, *p*)—maximum output flow rate that can enter the merge point from on-ramp *i* during time step *t* in time interval *p*; it is constrained by Lane 1 (shoulder lane) flow on segment *i* and the segment *i* capacity or by a queue spillback filling the mainline segment from a bottleneck further downstream, whichever governs.
- *ONRQ*(*i*, *t*, *p*)—unmet demand that is stored as a queue on the on-ramp roadway at node *i* during time step *t* in time interval *p* (veh).
- *RM*(*i*, *p*)—maximum allowable rate of an on-ramp meter at the on-ramp at node *i* during time interval *p* (veh/h).

OFF-RAMP VARIABLES

- *DEF*(*i*, *t*, *p*)—deficit: unmet demand from a previous time interval *p* that flows past node *i* during time step *t*; it is used in off-ramp flow calculations downstream of a bottleneck.
- *OFRD*(*i*, *p*)—desired off-ramp demand flow exiting at off-ramp *i* during time interval *p*.
- *OFRF*(*i*, *t*, *p*)—actual flow that can exit at off-ramp *i* during time step *t* in time interval *p*.

FACILITYWIDE VARIABLES

- *K*(*NS*, *P*)—average vehicle density over the entire facility during the entire analysis period *P*.
- *K*(*NS*, *p*)—average vehicle density over the entire facility during time interval *p*.
- *SMS*(*NS*, *P*)—average analysis period facility speed: average space mean speed over the entire facility during the entire analysis period *P*.

- $SMS(NS, p)$—average time interval facility speed: average space mean speed over the entire facility during time interval p.

TRAVEL TIME RELIABILITY VARIABLES

- CR_j—crash rate per 100 million vehicle miles traveled (VMT) in month j.
- D_{SP}—duration of study period SP (h).
- $DAF_s(tp, seg)$—demand adjustment factor for scenario s, period tp, and segment seg.
- DC_s—demand combination associated with scenario s.
- $DM(s)$—demand multiplier associated with scenario s.
- DM(Seed)—demand multiplier associated with the seed file.
- $\overline{DM}_j$—weighted average demand multiplier for all days in month j relative to seed value.
- $E[n_w, j]$—expected frequency of weather event w in month j, rounded to the nearest integer.
- $E_{15min}[D_w]$—expected duration of weather event w, rounded to the nearest 15-min increment.
- $\mathbb{G}(i)$—distribution function for incident with severity type i.
- ICR—incident-to-crash ratio.
- Inc_{Dur}—incident duration (min).
- Inc_{Type}—incident severity type (1–5).
- n_{inc}—number of incidents.
- n_j—expected frequency of all incidents in the study period for month j, rounded to the nearest integer.
- $n_{\text{Day},k}$—number of days in the reliability reporting period associated with demand combination k.
- N_{DC}—number of demand-level combinations considered.
- N_{Scen}—number of scenarios in the analysis.
- $N_{Inc,i}$—number of incidents associated with severity type i.
- $N_{Scen,Inc}$—number of all incident events generated for all scenarios.
- $N_{Scen,j}$—number of scenarios associated with month j of the reliability reporting period.
- $\bar{N}_{DC,WZ}$—adjusted number of replications of a demand combination for which the work zone is active.
- $P\{s\}$— probability of scenario s.
- $P_t\{w, j\}$—time-wise probability of weather type w in month j.

- r_{DC}—ratio of weekday types with an active work zone in a given month to the total number of each weekday type occurring in a given month.
- VMT_{Seed}—vehicle miles of travel in the seed file.
- $VMT_{seg,u}$—vehicle miles traveled on segment *seg* during analysis period *u* in the seed file.
- δ_x—adjustment parameters to satisfy equilibrium calibration equations.

3. UNDERSATURATED SEGMENT EVALUATION

FACILITY SPEED CONSTRAINT

This module begins with the first segment in the first time interval. For each cell, the flow (or volume) is equal to demand, the volume-to-capacity ratio is equal to the demand-to-capacity ratio, and undersaturated flow conditions prevail. Performance measures for the first segment during the first time interval are calculated by using the procedures for the corresponding segment type in Chapters 12, 13, and 14.

The analysis continues to the next downstream freeway segment in the same time interval, and the performance measures are calculated. The process is continued until the final downstream freeway segment cell in this time interval has been analyzed. For each cell, the volume-to-capacity ratio and performance measures are calculated for each freeway segment in the first time interval. The analysis continues in the second time interval beginning at the furthest upstream freeway segment and moving downstream until all freeway segments in that time interval have been analyzed. This pattern continues for the third time interval, fourth time interval, and so on until the methodology encounters a time interval that contains one or more segments with a demand-to-capacity ratio greater than 1.00 or when the final segment in the final time interval is analyzed. If no oversaturated segments are encountered, the segment performance measures are taken directly from Chapters 12, 13, and 14, and the facility performance measures are calculated as described next in the Directional Facility Module subsection.

On the facility level, a speed constraint is introduced that limits the maximum achievable speed downstream of a segment experiencing a low average speed.

When the analysis moves from isolated segments to a facility, an additional constraint is necessary that controls the relative speed between two segments. To limit the speeds downstream of a segment experiencing a low average speed, a maximum achievable speed is imposed on the downstream segments. This maximum speed is based on acceleration characteristics reported elsewhere (*4*) and is shown in Equation 25-1.

Equation 25-1

$$V_{max} = FFS - (FFS - V_{prev}) \times e^{-0.00162 \times L}$$

where

V_{max} = maximum achievable segment speed (mi/h),

FFS = segment free-flow speed (mi/h),

V_{prev} = average speed on immediate upstream segment (mi/h), and

L = distance from midpoints of the upstream segment and the subject segment (ft).

DIRECTIONAL FACILITY MODULE

The traffic performance measures can be aggregated over the length of the directional freeway facility, over the time duration of the study interval, or over the entire time–space domain. Each measure is discussed in the following paragraphs.

Aggregating the estimated traffic performance measures over the entire length of the freeway facility provides facilitywide estimates for each time interval. Facilitywide travel times, vehicle distance of travel, and vehicle hours of travel and delay can be computed, and patterns of their variation over the connected time intervals can be assessed. The computational engine is limited to 15-min time intervals and 1-min time steps.

Aggregating the estimated traffic performance measures over the time duration of the study interval provides an assessment of the performance of each segment along the freeway facility. Average and cumulative distributions of speed and density for each segment can be determined, and patterns of the variation over connected freeway segments can be compared. Average trip times, vehicle distance of travel, and vehicle hours of travel are easily assessed for each segment and compared.

Aggregating the estimated traffic performance measures over the entire time–space domain provides an overall assessment over the study interval time duration. Overall average speeds, average trip times, total vehicle distance traveled, and total vehicle hours of travel and delay are the most obvious overall traffic performance measures. Equation 25-2 through Equation 25-5 show how the facilitywide performance measures are calculated.

Facility space mean speed in time interval p is calculated with Equation 25-2:

Equation 25-2

$$SMS_p(NS) = \frac{\sum_{i=1}^{NS} SF(i,p) \times L(i)}{\sum_{i=1}^{NS} SF(i,p) \times \frac{L(i)}{U(i,p)}}$$

Average facility density in time interval p is calculated with Equation 25-3:

Equation 25-3

$$K_p(NS) = \frac{\sum_{i=1}^{NS} K(i,p) \times L(i)}{\sum_{i=1}^{NS} L(i) \times N(i,p)}$$

Overall space mean speed across all intervals is calculated with Equation 25-4:

Equation 25-4

$$SMS(NS,p) = \frac{\sum_{p=1}^{P} \sum_{i=1}^{NS} SF(i,p) \times L(i)}{\sum_{p=1}^{P} \sum_{i=1}^{NS} SF(i,p) \times \frac{L(i)}{U(i,p)}}$$

Overall average density across all intervals is calculated with Equation 25-5:

Equation 25-5

$$K(NS,p) = \frac{\sum_{p=1}^{P} \sum_{i=1}^{NS} K(i,p) \times L(i)}{\sum_{p=1}^{P} \sum_{i=1}^{NS} L(i) \times N(i,p)}$$

These performance measures can be compared for different alternatives to assess the impacts of different volume scenarios or the effects of geometric improvements to the facility.

4. OVERSATURATED SEGMENT EVALUATION

Oversaturated flow conditions occur when the demand on one or more freeway segment cells exceeds its capacity. The oversaturated segment evaluation procedure presented in this chapter is performed separately for general purpose and managed lanes. To evaluate the effect of interactions between the general purpose and managed lanes, additional delays are introduced and calculated in the form of vertical queueing, which is discussed at the end of this section.

Once oversaturation is encountered, the methodology changes its temporal and spatial units of analysis. The spatial units become nodes and segments, and the temporal unit moves from a time interval to smaller time steps. A node is defined as the junction of two segments. There is always one more node than there are segments, with a node added at the beginning and end of each segment. The numbering of nodes and segments begins at the upstream end and moves to the downstream end, with the segment upstream of node i numbered segment $i - 1$ and the downstream segment numbered i, as shown in Exhibit 25-1. The intermediate segments and node numbers represent the division of the section between Ramps 1 and 2 into three segments numbered 2 (ONR), 3 (BASIC), and 4 (OFR). The oversaturated analysis moves from the first node to each downstream node in the same time step. After completion of a time step, the same nodal analysis is performed for subsequent time steps.

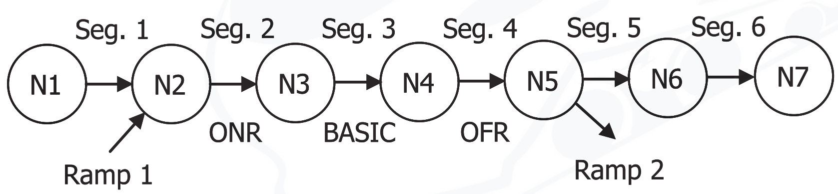


Exhibit 25-1
Node–Segment Representation of a Directional Freeway Facility

The oversaturated analysis focuses on the computation of segment average flows and densities in each time interval. These parameters are later aggregated to produce facilitywide estimates. Two key inputs into the flow estimation procedures are the time step duration for flow updates and a flow–density function. These two inputs are described in the next subsections.

PROCEDURE PARAMETERS

Time Step Duration

Segment flows are calculated in each time step and are used to calculate the number of vehicles on each segment at the end of every time step. The number of vehicles on each segment is used to track queue accumulation and discharge and to calculate the average segment density.

To provide accurate estimates of flows in oversaturated conditions, the time intervals are divided into smaller time steps. The conversion from time intervals to time steps occurs during the first oversaturated time interval and remains until the end of the analysis. The transition to time steps is essential because, at certain points in the methodology, future performance estimates are made on the basis of the past value of a variable.

The computational engine assumes a time step of 15 s for oversaturated flow computations, which is adequate for most facilities with a minimum segment length greater than 300 ft. This time step is based on the assumption that a shockwave of (severe) congestion can travel at speeds up to 20 ft/s or 13.6 mi/h. A minimum segment length of 300 ft ensures that the congestion shockwave does not travel more than one segment length in one 15-s time step.

For shorter segments, two problem situations may arise. The first situation occurs when segments are short and the rate of queue growth (shockwave speed) is rapid. Under these conditions, a short segment may be completely undersaturated in one time step and completely queued in another. The methodology may store more vehicles in this segment during a time step than space allows. Fortunately, the next time step compensates for this error, and the procedure continues to track queues and store vehicles accurately after this correction.

The oversaturated methodology implemented in the computational engine assumes a time step of 15 s, which is adequate for segment lengths greater than 300 ft.

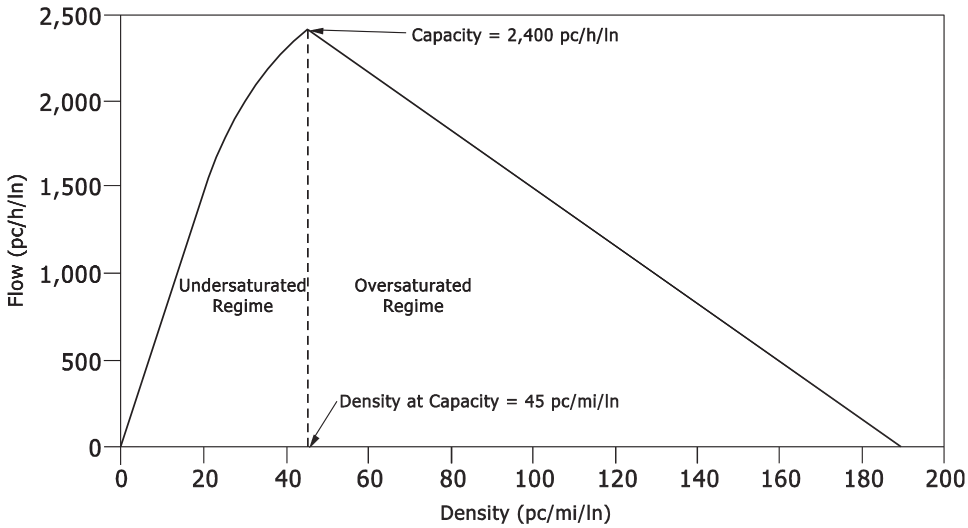
The second situation in which small time steps are important occurs when two queues interact. There is a temporary inaccuracy due to the maximum output of a segment changing, thus causing the estimation of available storage to be slightly in error. This situation results in the storage of too many vehicles on a particular segment. This "supersaturation" is temporary and is compensated for in the next time step. Inadequate time step size will result in erroneous estimation of queue lengths and may affect other performance measures as well. Regardless, if queues interact, the results should be viewed with extreme caution.

Flow–Density Relationship

Analysis of freeway segments depends on the relationships between segment speed, flow, and density. Chapter 12, Basic Freeway and Multilane Highway Segments, defines a relationship between these variables and the calculation of performance measures in the undersaturated regime. The freeway facilities methodology presented here uses the same relationships for undersaturated segments. In other words, when a segment is undersaturated the computations of this methodology are identical to the results obtained from Chapters 12, 13, and 14 for basic freeway segments, weaving segments, and ramp segments, respectively.

The calculations for oversaturated segments assume a simplified linear flow–density diagram in the congested region. Exhibit 25-2 shows this flow–density diagram for a segment having a free-flow speed (FFS) of 75 mi/h. For other FFSs, the corresponding capacities in Chapters 12, 13, and 14 should be used.

The oversaturated regime curve in Exhibit 25-2 is constructed from a user-specified jam density (default is 190 pc/mi/ln) and the known value of capacity, defined as the flow at a density of 45 pc/mi/ln. The flow–density relationship is assumed to be linear between these two points. The slope of the resulting line describes the speed of the shock wave at which queues grow and dissipate, as discussed further below. The speed in a congested segment is obtained from the prevailing density in the segment, read along the linear flow–density relationship. Details on the theory of kinematic waves in highway traffic are given elsewhere (*5*–*7*).



Note: Assumed FFS = 75 mi/h.

Exhibit 25-2
Segment Flow–Density Function

FLOW ESTIMATION

The oversaturated portion of the methodology is detailed as a flowchart in Exhibit 25-3. The flowchart is divided into several sections over several pages. Processes that continue from one section of the flowchart to another are indicated by capital letters within parallelograms. Computations are detailed and labeled in the subsections that follow according to each step of the flowchart.

The procedure first calculates flow variables starting at the first node during the first time step of oversaturation and followed by each downstream node and segment in the same time step. After all computations in the first time step are completed, calculations are performed at each node and segment during subsequent time steps for all remaining time intervals until the analysis is completed.

Exhibit 25-3
Oversaturated Analysis Procedure

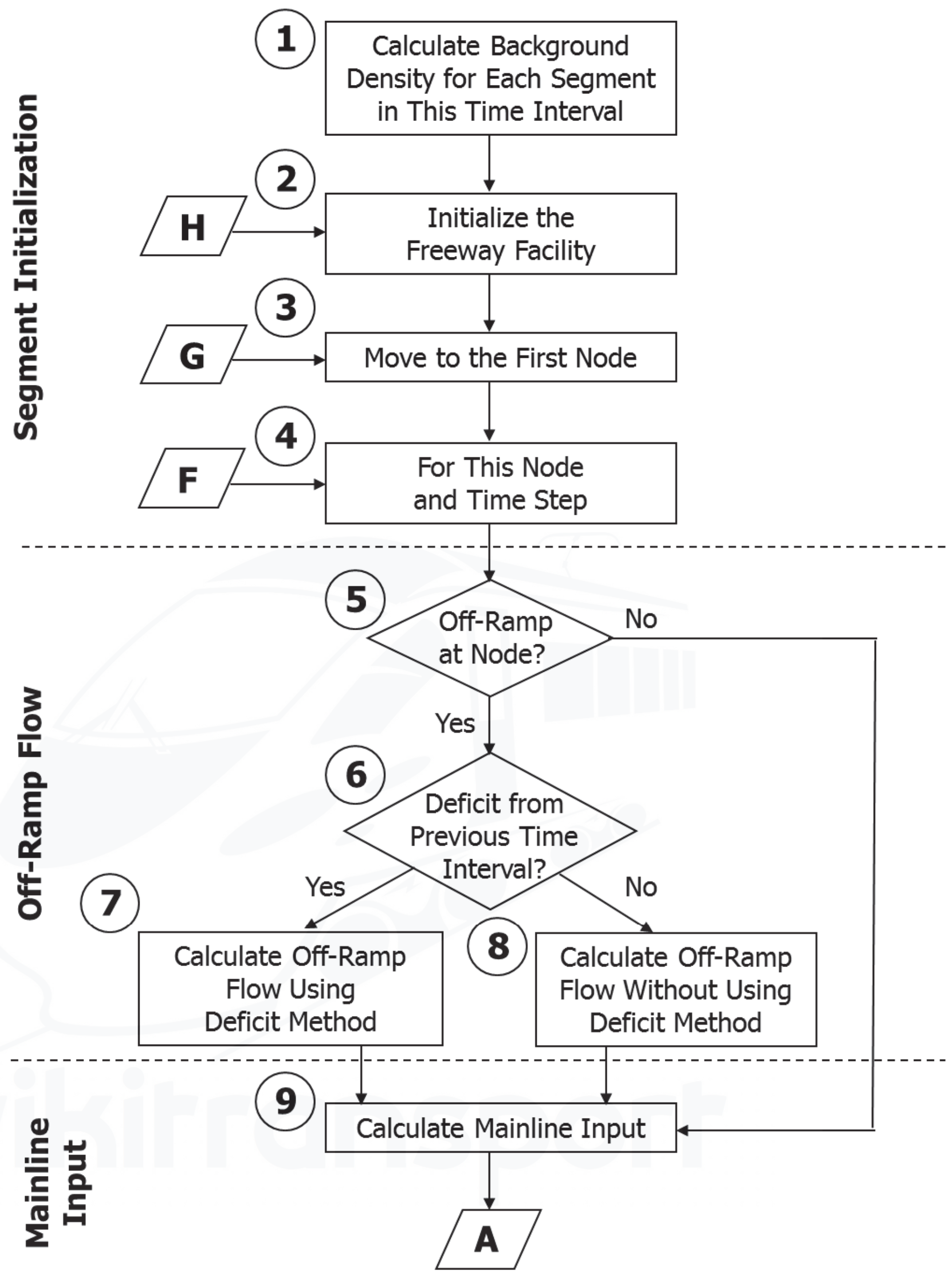


Exhibit 25-3 (cont'd.)
Oversaturated Analysis Procedure

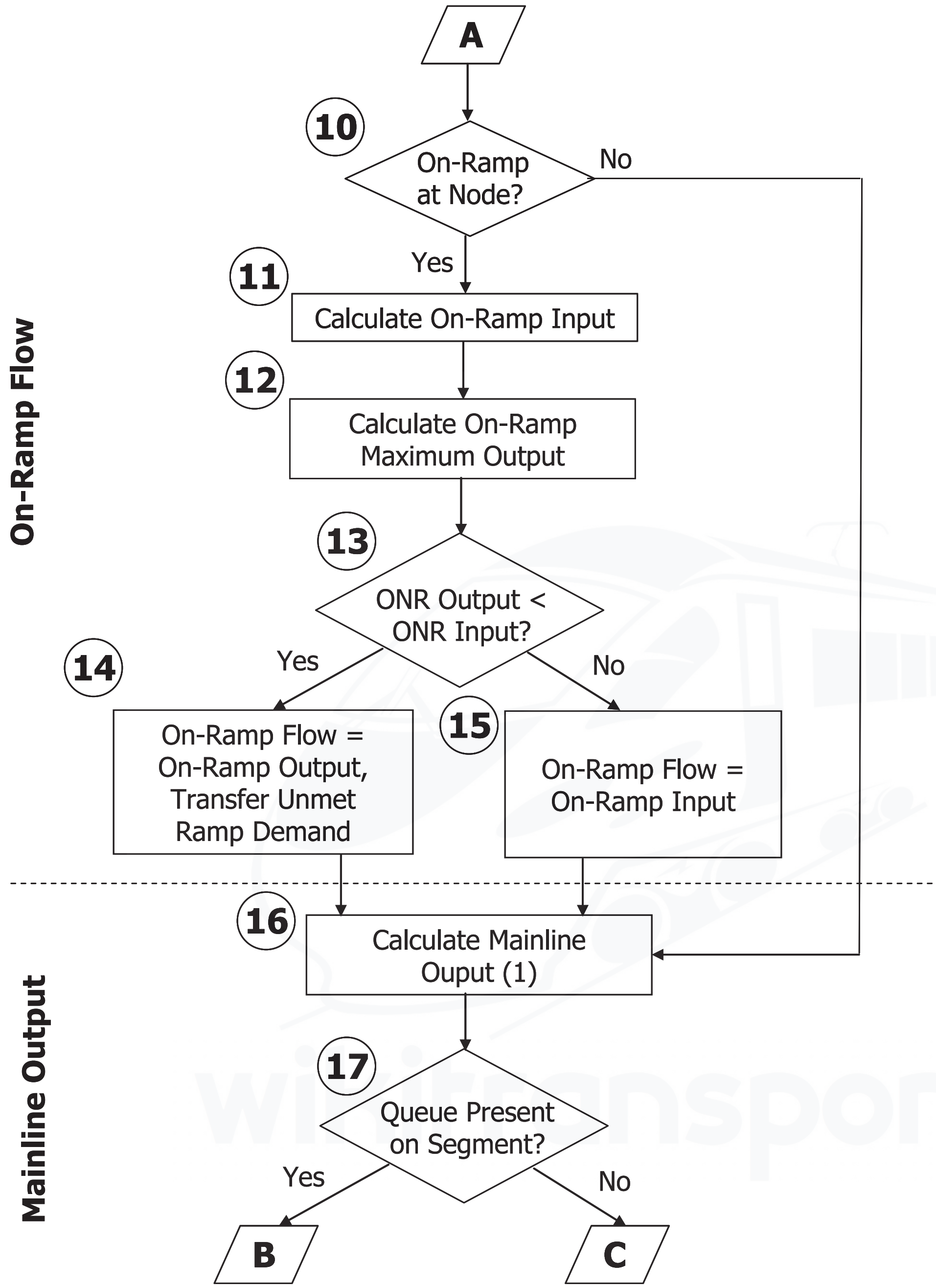


Exhibit 25-3 (cont'd.)
Oversaturated Analysis Procedure

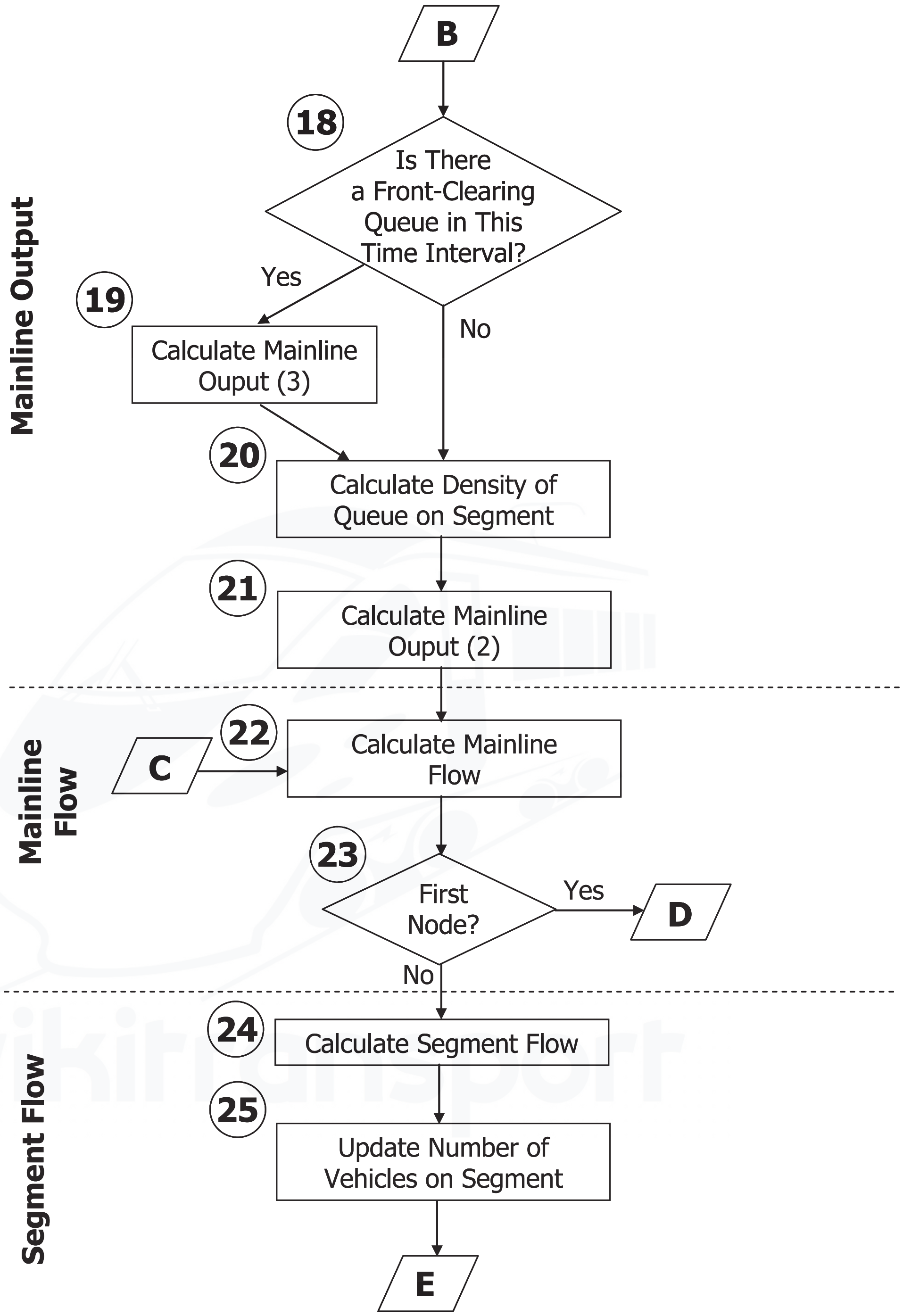
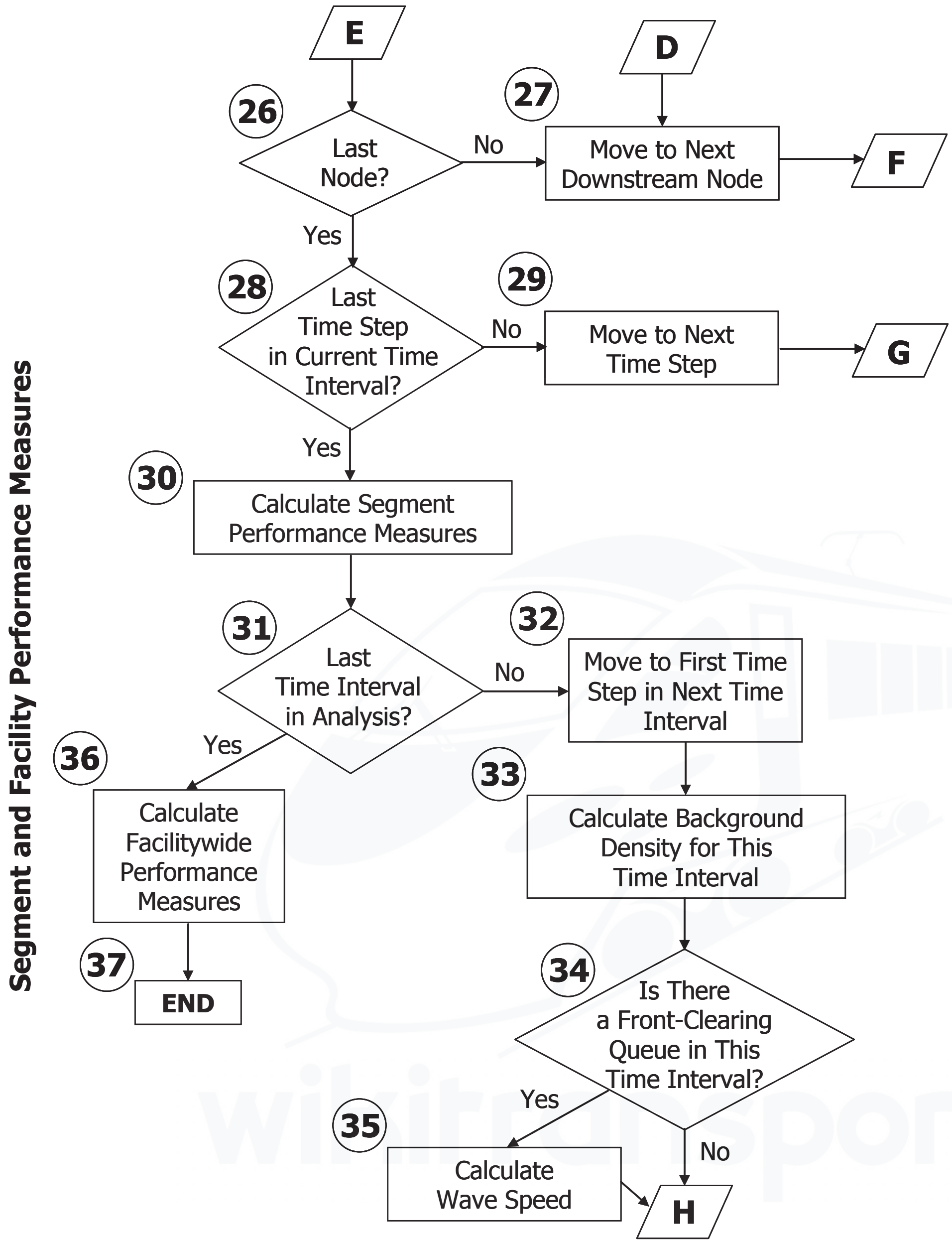


Exhibit 25-3 (cont'd.) Oversaturated Analysis Procedure



Segment Initialization: Exhibit 25-3, Steps 1–4

Steps 1–4 of the oversaturated procedure prepare the flow calculations for the first time step and specify return points for later time steps. To calculate the number of vehicles on each segment at the various time steps, the segments must contain the proper number of vehicles before the queuing analysis places unserved vehicles on segments. The initialization of each segment is described below. A simplified queuing analysis is initially performed to account for the effects of upstream bottlenecks. These bottlenecks meter traffic downstream of their location. The storage of unserved vehicles (those unable to enter the bottleneck) on upstream segments is performed in a later module. To obtain the proper number of vehicles on each segment, the expected demand *ED* is calculated. Expected demand is based on demands for and capacities of the segment and includes the effects of all upstream segments. The expected demand is the flow of traffic expected to arrive at each segment if all queues were stacked vertically (i.e., no upstream effects of queues). In other words, all segments upstream of a bottleneck have expected demands equal to their actual demand. The expected demand of the bottleneck segment and all further downstream segments is calculated by assuming a capacity constraint at the bottleneck, which meters traffic to downstream segments. The expected demand ED is calculated for each segment with Equation 25-6:

Equation 25-6

$$ED(i,p) = \min[SC(i,p), ED(i-1,p) + ONRD(i,p) - OFRD(i,p)]$$

The segment capacity SC applies to the length of the segment. With the expected demand calculated, the background density KB can be obtained for each segment by using the appropriate segment density estimation procedures in Chapters 12, 13, and 14. The background density is used to calculate the number of vehicles NV on each segment by using Equation 25-7. If there are unserved vehicles at the end of the preceding time interval, the unserved vehicles UV are transferred to the current time interval. Here, S refers to the final time step in the preceding time interval. The (0) term in NV represents the start of the first time step in time interval p. The corresponding term at the end of the time step is $NV(i, 1, p)$.

Equation 25-7

$$NV(i,0,p) = KB(i,p) \times L(i) + UV(i,S,p-1)$$

The number of vehicles calculated from the background density is the minimum number of vehicles that can be on the segment at any time. This constraint is a powerful check on the methodology because the existence of queues downstream cannot reduce this minimum. Rather, the segment can only store additional vehicles. The storage of unserved vehicles is determined in the segment flow calculation module later in this chapter.

Mainline Flow Calculations: Exhibit 25-3, Steps 9 and 16–23

The description of ramp flows follows the description of mainline flows. Thus, Steps 5–8 and 10–15 are skipped at this time to focus first on mainline flow computations. Because of skipping steps in the descriptions, some computations may include variables that have not been described but that have already been calculated in the flowchart.

Flows analyzed in oversaturated conditions are calculated for every time step and are expressed in terms of vehicles per time step. The procedure separately analyzes the flow across a node on the basis of the origin and destination of the flow across the node. The mainline flow is defined as the flow passing from upstream segment $i - 1$ to downstream segment i. It does not include the on-ramp flow. The flow to an off-ramp is the off-ramp flow. The flow from an on-ramp is the on-ramp flow. Each of these flows is shown in Exhibit 25-4 with the origin, destination, and relationship to segment i and node i.

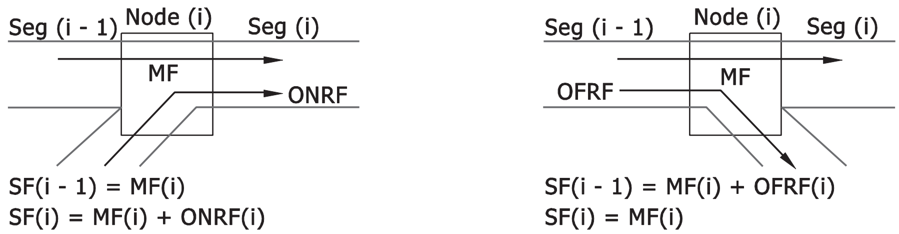


Exhibit 25-4
Definitions of Mainline and Segment Flows

The segment flow is the total output of a segment, as shown in Exhibit 25-4. Segment flows are calculated by determining the mainline and ramp flows. The mainline flow is calculated as the minimum of six constraints: mainline input (*MI*), *MO*1, *MO*2, *MO*3, upstream segment $i - 1$ capacity, and downstream segment i capacity, as explained next.

Mainline Input: Exhibit 25-3, Step 9

Mainline input *MI* is the number of vehicles that wish to travel through a node during the time step. The calculation includes (*a*) the effects of bottlenecks upstream of the analysis node, (*b*) the metering of traffic during queue accumulation, and (*c*) the presence of additional traffic during upstream queue discharge.

MI is calculated by taking the number of vehicles entering the node upstream of the analysis node, adding on-ramp flows or subtracting off-ramp flows, and adding the number of unserved vehicles on the upstream segment. Thus, *MI* is the maximum number of vehicles that wish to enter a node during a time step. *MI* is calculated by using Equation 25-8, where all values have units of vehicles per time step.

Equation 25-8

$$\begin{aligned} MI(i,t,o) = {} & MF(i-1,t,p) + ONRF(i-1,t,p) - OFRF(i,t,p) \\ & + UV(i-1,t-1,p) \end{aligned}$$

Mainline Output: Exhibit 25-3, Steps 16–21

The mainline output is the maximum number of vehicles that can exit a node, constrained by downstream bottlenecks or by merging on-ramp traffic. Different constraints on the output of a node result in three separate types of mainline outputs (*MO*1, *MO*2, and *MO*3).

Mainline Output 1, Ramp Flows: Exhibit 25-3, Step 16

*MO*1 is the constraint caused by the flow of vehicles from an on-ramp. The capacity of an on-ramp segment is shared by two competing flows. This on-ramp flow limits the flow from the mainline through this node. The total flow that can pass the node is estimated as the minimum of the segment i capacity and the mainline outputs from the preceding time step. The sharing of Lane 1 (shoulder lane) capacity is determined in the calculation of the on-ramp. *MO*1 is calculated by using Equation 25-9.

Equation 25-9

$$MO1 = \min \begin{cases} SC(i,t,p) - ONRF(i,t,p) \\ MO2(i,t-1,p) \\ MO3(i,t-1,p) \end{cases}$$

Mainline Output 2, Segment Storage: Exhibit 25-3, Steps 20 and 21

The second constraint on the output of mainline flow through a node is caused by the growth of queues on a downstream segment. As a queue grows on a segment, it may eventually limit the flow into the current segment once the boundary of the queue reaches the upstream end of the segment. The boundary of the queue is treated as a shock wave. $MO2$ is a limit on the flow exiting a node due to the presence of a queue on the downstream segment.

The $MO2$ limitation is determined first by calculating the maximum number of vehicles allowed on a segment at a given queue density. The maximum flow that can enter a queued segment is the number of vehicles that leave the segment plus the difference between the maximum number of vehicles allowed on the segment and the number of vehicles already on the segment. The density of the queue is calculated by using Equation 25-10 for the linear density–flow relationship shown in Exhibit 25-2 earlier.

Equation 25-10

$$KQ(i,t,p) = KJ - [(KJ - KC) \times SF(i,t-1,p)]/SC(i,p)$$

Once the queue density is computed, $MO2$ can be computed by using Equation 25-11.

Equation 25-11

$$MO2\ (i,t,p) = SF(i,t-1,p) - ONRF(i,t,p) + [KQ(i,t,p) \times L(i)] - NV(i,t-1,p)$$

The performance of the downstream node is estimated by taking the performance during the preceding time step. This estimation remains valid when there are no interacting queues. When queues interact and the time steps are small enough, the error in the estimations is corrected in the next time step.

Mainline Output 3, Front-Clearing Queues: Exhibit 25-3, Steps 17–19

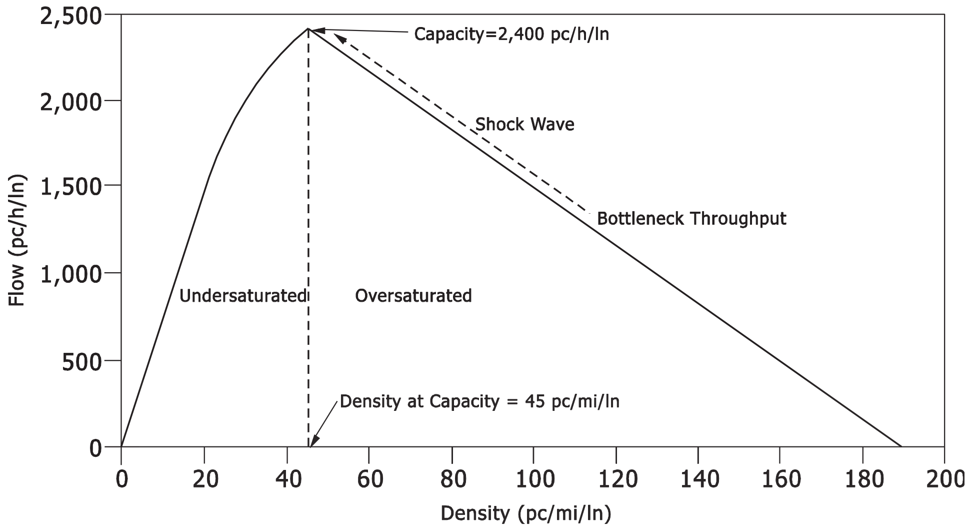
The final constraint on exiting mainline flows at a node is caused by downstream queues clearing from their downstream end. These front-clearing queues are typically caused by incidents in which there is a temporary reduction in capacity. A queue will clear from the front if two conditions are satisfied. First, the segment capacity (minus the on-ramp demand if present) for this time interval must be greater than the segment capacity (minus the ramp demand if present) in the preceding time interval. The second condition is that the segment capacity minus the ramp demand for this time interval must be greater than the segment demand for this time interval. A queue will clear from the front if both conditions in the following inequality (Equation 25-12) are met.

Equation 25-12

$$\text{If } [SC(i,p) - ONRD(i,p)] > [SC(i,p-1) - ONRD(i,p-1)]$$
$$\text{and } [SC(i,p) - ONRD(i,p)] > SD(i,p)$$

A segment with a front-clearing queue will have the number of vehicles stored decrease during recovery, while the back of the queue position is unaffected. Thus, the clearing does not affect the segment throughput until the recovery wave has reached the upstream end of the front-clearing queue. The computational engine implementation is simplified by assuming the downstream segment is fully queued when the $MO3$ constraint is applied. In the flow–density graph shown in Exhibit 25-5, the wave speed is estimated by the slope of the dashed line connecting the bottleneck throughput and the segment capacity points.

Exhibit 25-5
Flow–Density Function with a Shock Wave



Note: Assumed FFS = 75 mi/h.

The assumption of a linear flow–density function greatly simplifies the calculation of the wave speed. The bottleneck throughput value is not required to estimate the speed of the shock wave that travels along a known line. All that is required is the slope of the line, which is calculated with Equation 25-13.

$$WS(i,p) = SC(i,p)/[N(i,p) \times (KJ - KC)]$$

Equation 25-13

The wave speed is used to calculate the wave travel time *WTT*, which is the time it takes the front queue-clearing shock wave to traverse this segment. Dividing the wave speed *WS* by the segment length in miles gives *WTT*.

The recovery wave travel time is the time required for the conditions at the downstream end of the current segment to reach the upstream end of the current segment. To place a limit on the current node, the conditions at the downstream node are observed at a time in the past. This time is the wave travel time. This constraint on the current node is *MO*3. The calculation of *MO*3 uses Equation 25-14 and Equation 25-15. If the wave travel time is not an integer number of time steps, then the weighted average performance of each variable is taken for the time steps nearest the wave travel time. This method is based on a process described elsewhere (*5*–*7*).

$$WTT = T \times L(i)/WS(i,p)$$

Equation 25-14

$$MO3\ (i,t,p) = \min \left\{ \begin{array}{c} MO1\ (i+1, t-WTT, p) \\ MO2\ (i+1, t-WTT, p) + OFRF(i+1, t-WTT, p) \\ MO3\ (i+1, t-WTT, p) + OFRF(i+1, t-WTT, p) \\ SC(i, t-WTT, p) \\ SC(i+1, t-WTT, p) + OFRF(i+1, t-WTT, p) \end{array} \right\} - OFRF(i,t,p)$$

Equation 25-15

Mainline Flow: Exhibit 25-3, Steps 22 and 23

The flow across a node is called the mainline flow *MF* and is the minimum of the following variables: *MI*, *MO*1, *MO*2, *MO*3, upstream segment $i-1$ capacity, and downstream segment *i* capacity, as shown in Equation 25-16.

Equation 25-16

$$MF(i,t,p) = \min \left\{ \begin{array}{c} MI(i,t,p) \\ MO1\,(i,t,p) \\ MO2\,(i,t,p) \\ MO3\,(i,t,p) \\ SC(i,t,p) \\ SC(i-1,t,p) \end{array} \right\}$$

In addition to mainline flows, ramp flows must be analyzed. The presence of mainline queues also affects ramp flows.

On-Ramp Calculations: Exhibit 25-3, Steps 10–15

On-Ramp Input: Exhibit 25-3, Steps 10 and 11

The maximum on-ramp input *ONRI* is calculated by adding the on-ramp demand and the number of vehicles queued on the ramp. The queued vehicles are treated as unmet ramp demand that was not served in previous time steps. The on-ramp input is calculated with Equation 25-17.

Equation 25-17

$$ONRI(i,t,p) = ONRD(i,t,p) + ONRQ(i,t-1,p)$$

On-Ramp Output: Exhibit 25-3, Step 12

The maximum on-ramp output *ONRO* is calculated on the basis of the mainline traffic through the node where the on-ramp is located. The on-ramp output is the minimum of two values. The first is segment i capacity minus *MI*, in the absence of downstream queues. Otherwise, the segment capacity is replaced by the throughput of the queue. This estimation implies that vehicles entering an on-ramp segment will fill Lanes 2 to N (where N is the number of lanes on the current segment) to capacity before entering Lane 1. This assumption is consistent with the estimation of v_{12} from Chapter 14, Freeway Merge and Diverge Segments.

The second case occurs when the Lane 1 flow on segment i is greater than one-half of the Lane 1 capacity. At this point, the on-ramp maximum output is set to one-half of Lane 1 capacity. This output limitation implies that when the demands from the freeway and the on-ramp are very high, there will be forced one-to-one merging on the freeway from the freeway mainline and the on-ramp in Lane 1. An important characteristic of traffic behavior is that, in a forced merging situation, ramp and right-lane freeway vehicles will generally merge one on one, sharing the capacity of the rightmost freeway lane (*8*). In all cases, the on-ramp maximum output is also limited to the physical ramp road capacity and the ramp-metering rate, if present. The maximum on-ramp output is an important limitation on the ramp flow. Queuing occurs when the combined demand from the upstream segment and the on-ramp exceeds the throughput of the ramp segment. The queue can be located on the upstream segment, on the ramp, or on both and depends on the on-ramp maximum output. Equation 25-18 determines the value of the maximum on-ramp output.

$$ONRO(i,t,p) = \min\left\{\begin{matrix} RM(i,t,p) \\ ONRC(i,t,p) \\ \max\left\{\begin{matrix} \min\left\{\begin{matrix} SC(i,t,p) \\ MF(i+1,t-1,p) + ONRF(i,t-1,p) \\ MO3\,(i,t-1,p) + ONRF(i,t-1,p) \end{matrix}\right\} - MI(i,t,p) \\ \min\left\{\begin{matrix} SC(i,t,p) \\ MF(i+1,t-1,p) + ONRF(i,t-1,p) \\ MO3\,(i,t-1,p) + ONRF(i,t-1,p) \end{matrix}\right\} / 2N(i,p) \end{matrix}\right\} \end{matrix}\right\}$$

Equation 25-18

This model incorporates the maximum mainline output constraints from downstream queues, not just the segment capacity. This fact is significant because as a queue spills over an on-ramp segment, the flow through Lane 1 is constrained. This constraint, in turn, limits the flow that can enter Lane 1 from the on-ramp. The values of $MO2$ and $MO3$ for this time step are not yet known, so they are estimated from the preceding time step. This estimation is one rationale for using small time steps. If there is forced merging during the time step when the queue spills back over the current node, the on-ramp will discharge more than its share of vehicles (i.e., more than 50% of the Lane 1 flow). This situation will cause the mainline flow past node i to be underestimated. But during the next time step, the on-ramp flow will be at its correct flow rate, and a one-to-one sharing of Lane 1 will occur.

On-Ramp Flows, Queues, and Delays: Exhibit 25-3, Steps 13–15

Finally, the on-ramp flow is calculated on the basis of the on-ramp input and output values computed above. If the on-ramp input is less than the on-ramp output, then the on-ramp demand can be fully served in this time step and Equation 25-19 is used.

$$ONRF(i,t,p) = ONRI(i,t,p)$$

Equation 25-19

Otherwise, the ramp flow is constrained by the maximum on-ramp output, and Equation 25-20 is used.

$$ONRF(i,t,p) = ONRO(i,t,p)$$

Equation 25-20

In the latter case, the number of vehicles in the ramp queue is updated by using Equation 25-21.

$$ONRQ(i,t,p) = ONRI(i,t,p) - ONRO(i,t,p)$$

Equation 25-21

The total delay for on-ramp vehicles can be estimated by integrating the value of on-ramp queues over time. The methodology uses the discrete queue lengths estimated at the end of each interval $ONRQ(i, S, p)$ to produce overall ramp delays by time interval.

Off-Ramp Flow Calculation: Exhibit 25-3, Steps 5–8

The off-ramp flow is determined by calculating a diverge percentage on the basis of the segment and off-ramp demands. The diverge percentage varies only by time interval and remains constant for vehicles that are associated with a particular time interval. If there is an upstream queue, traffic may be metered to this off-ramp, which will cause a decrease in the off-ramp flow. When the vehicles that were metered arrive in the next time interval, they use the diverge percentage associated with the preceding time interval. A deficit in flow, caused by traffic from an upstream queue meter, creates delays for vehicles destined to this off-ramp and other downstream destinations. The upstream segment flow is used because the procedure assumes a vehicle destined for an off-ramp is able to exit at the off-ramp once it enters the off-ramp segment. This deficit is calculated with Equation 25-22.

Equation 25-22

$$DEF(i,t,p) = \max\left\{ \begin{array}{c} 0 \\ \left(\sum_{X=1}^{p-1} SD(i-1,X) - \sum_{X=1}^{p-1}\sum_{t=1}^{T}[MF(i-1,t,X) + ONRF(i-1,t,X)] + \sum_{t=1}^{t-1}[MF(i-1,t,p) + ONRF(i-1,t,p)] \right) \end{array} \right\}$$

If there is a deficit, then the off-ramp flow is calculated by using the deficit method. The deficit method is used differently in two specific situations. If the upstream mainline flow plus the flow from an on-ramp at the upstream node (if present) is less than the deficit for this time step, then the off-ramp flow is equal to the mainline and on-ramp flows times the off-ramp turning percentage in the preceding time interval, as indicated in Equation 25-23.

Equation 25-23

$$OFRF(i,t,p) = [MF(i-1,t,p) + ONRF(i-1,t,p)] \times \left[\frac{OFRD(i,p-1)}{SD(i-1,p-1)}\right]$$

However, if the deficit is less than the upstream mainline flow plus the on-ramp flow from an on-ramp at the upstream node (if present), then Equation 25-24 is used. This equation separates the flow into the remaining deficit flow and the balance of the arriving flow.

Equation 25-24

$$OFRF(i,t,p) = DEF(i,t,p) \times \left[\frac{OFRD(i,p-1)}{SD(i-1,p-1)}\right] + [MF(i-1,t,p) + ONRF(i-1,t,p) - DEF(i,t,p)] \times \left[\frac{OFRD(i,p)}{SD(i-1,p)}\right]$$

If there is no deficit, then the off-ramp flow is equal to the sum of the upstream mainline flow plus the on-ramp flow from an on-ramp at the upstream node (if present) multiplied by the off-ramp turning percentage for this time interval according to Equation 25-25.

Equation 25-25

$$OFRF(i,t,p) = [MF(i-1,t,p) + ONRF(i-1,t,p)] \times \left[\frac{OFRD(i,p)}{SD(i-1,p)}\right]$$

The procedure does not incorporate any delay or queue length computations for off-ramps.

Segment Flow Calculation: Exhibit 25-3, Steps 24 and 25

The segment flow is the number of vehicles that flow out of a segment during the current time step. These vehicles enter the current segment either to the mainline or to an off-ramp at the current node. The vehicles that entered the upstream segment may or may not have become queued within the segment. The segment flow SF is calculated with Equation 25-26.

Equation 25-26

$$SF(i-1,t,p) = MF(i,t,p) + OFRF(i,t,p)$$

The number of vehicles on each segment is calculated on the basis of the number of vehicles that were on the segment in the preceding time step, the number of vehicles that entered the segment in this time step, and the number of vehicles that leave the segment in this time step. Because the number of vehicles that leave a segment must be known, the number of vehicles on the current segment cannot be determined until the upstream segment is analyzed. The number of vehicles on each segment NV is calculated with Equation 25-27.

$$NV(i-1,t,p) = NV(i-1,t-1,p) + MF(i-1,t,p) + ONRF(i-1,t,p) - MF(i,t,p) - OFRF(i,t,p)$$

Equation 25-27

The number of unserved vehicles stored on a segment is calculated as the difference between the number of vehicles on the segment and the number of vehicles that would be on the segment at the background density. The number of unserved vehicles UV stored on a segment is calculated with Equation 25-28.

$$UV(i-1,t,p) = NV(i-1,t,p) - [KB(i-1,p) \times L(i-1)]$$

Equation 25-28

If the number of unserved vehicles is greater than zero, then a queue is present on the facility upstream of the node in question. The presence of a queue and congestion indicates that the node capacity is in queue discharge mode, which means the queue discharge capacity is reduced relative to the pre-breakdown capacity by a factor α. To account for this queue discharge effect, Equation 25-29 is applied to any active bottleneck along the facility if $UV(i-1, t, p) > 0.001$. This tolerance over an absolute value of zero is necessary to account for potential rounding errors in the procedure.

$$SC(i,t,p) = (1-\alpha) \times SC(i,t,p)$$

Equation 25-29

SEGMENT AND RAMP PERFORMANCE MEASURES

In the final time step of a time interval, the segment flows are averaged over the time interval, and the performance measures for each segment are calculated. If there was no queue on a particular segment during the entire time interval, then the performance measures are calculated from the corresponding Chapter 12, 13, or 14 method for that segment. Because there are T time steps in an hour, the average segment flow rate in vehicles per hour in time interval p is calculated by using Equation 25-30.

$$SF(i,p) = \frac{T}{S}\sum_{t=1}^{S} SF(i,t,p)$$

Equation 25-30

If $T = 60$ (1-min time step) and $S = 15$ (interval = 15 min), then $T/S = 4$. If there was a queue on the current segment in any time step during the time interval, then the segment performance measures are calculated in three steps. First, the average number of vehicles NV over a time interval is calculated for each segment by using Equation 25-31.

$$NV(i,p) = \frac{1}{S}\sum_{t=1}^{S} NV(i,t,p)$$

Equation 25-31

Second, the average segment density K is calculated by taking the average number of vehicles NV for all time steps in the time interval and dividing it by the segment length, as shown by Equation 25-32.

Equation 25-32

$$K(i,p) = \frac{NV(i,p)}{L(i)}$$

Third, the average speed U on the current segment i during the current time interval p is calculated with Equation 25-33.

Equation 25-33

$$U(i,p) = \frac{SF(i,p)}{K(i,p)}$$

Additional segment performance measures can be derived from the basic measures shown in Equation 25-30 through Equation 25-33. Most prominent is segment delay, which can be computed as the difference in segment travel time at speed $U(i, p)$ and at the segment FFS.

The final segment performance measure is the length of the queue at the end of the time interval (i.e., step S in time interval p). The length of a queue Q on the segment, in feet, is calculated with Equation 25-34.

Equation 25-34

$$Q(i,p) = \frac{UV(i,S,p)}{\max\left[\left(KQ(i,S,p) - KB(i,p)\right), 1\right]} \times 5{,}280$$

OVERSATURATION ANALYSIS WITHIN MANAGED LANES

Whenever oversaturated conditions occur (as defined in Chapter 10) on freeway facilities that contain managed lanes, the freeway facilities methodology invokes the oversaturated analysis described in this chapter for both the general purpose and managed lane facilities. The analysis will be performed separately for each facility, meaning that the queues in either the general purpose or managed lanes do not interact with each other. For freeway facilities with managed lanes that do not have any access segments connecting the two lane groups, performing oversaturated analysis separately yields accurate performance measures for both the general purpose and managed lanes.

However, when access segments connect the two lane groups, no method currently exists to model the queue interaction between the two. In this situation, the queue spillback between the general purpose and managed lanes is modeled as a "vertical queue." The vehicles that cannot enter the general purpose or managed lane facilities due to the presence of a queue do not translate into actual queuing on the origin lane group, as shown in Exhibit 25-6.

The freeway facilities methodology keeps track of vehicles that cannot enter the downstream segment (past the access point) in the form of a vertical queue, and it releases these vehicles as congestion dissipates. Note that there are two vertical queues for each access segment, one for vehicles traveling from the managed to the general purpose lanes, and the other for vehicles traveling from the general purpose to the managed lanes. Exhibit 25-6 shows an example of a vertical queue for the first situation. Note that the existence of a vertical queue does not lead to actual queuing on the managed lane.

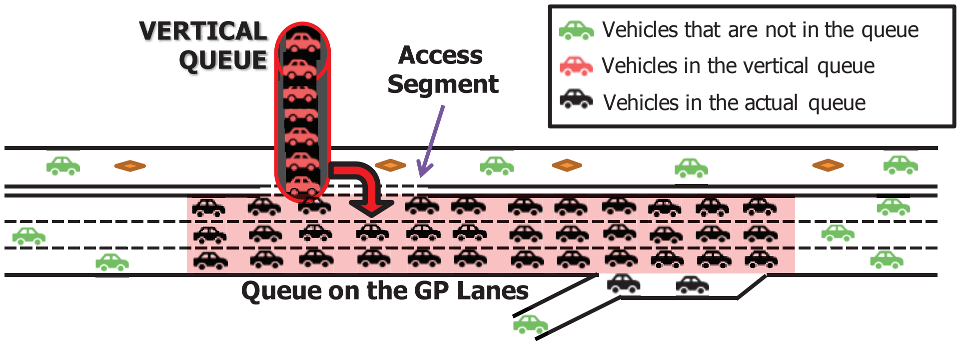


Exhibit 25-6
Vertical Queuing from a Managed Lane Due to Queue Presence on the General Purpose Lanes

Despite this simplification of queue spillback modeling, the methodology keeps track of the delays vehicles encounter in the vertical queues. The delay is computed as the number of vehicles stored in the vertical queue, multiplied by 15 min of delay in each analysis period. The delay of the vehicles originating from the managed lanes that are waiting in the vertical queue is estimated based on Equation 25-35.

Equation 25-35

$$D_{ML,vert} = N_{ML,vert} \times 0.25$$

where

$D_{ML,vert}$ = delay incurred by vehicles originating from the managed lanes waiting in the vertical queue for one 15-min analysis period (h) and

$N_{ML,vert}$ = average number of vehicles originating from the managed lanes that are waiting in the vertical queue in one analysis period (veh).

Similar to the vehicle delay in the managed lanes, the delay of vehicles originating from the general purpose lanes that are waiting in the vertical queue is estimated based on Equation 25-36.

Equation 25-36

$$D_{GP,vert} = N_{GP,vert} \times 0.25$$

where

$D_{GP,vert}$ = delay incurred by vehicles originating from the general purpose lanes waiting in the vertical queue for one 15-min analysis period (h) and

$N_{GP,vert}$ = average number of vehicles originating from the general purpose lanes that are waiting in the vertical queue in one analysis period (veh).

5. WORK ZONE ANALYSIS DETAILS

This section provides additional computational details for work zone analysis on freeway facilities. The analysis of work zones on basic segments on a facility is described in Chapter 10, Freeway Facilities Core Methodology; this section provides additional analysis details for work zones in merge, diverge, and weaving segments, as well as the analysis of directional crossover work zones. The information provided in this section is largely based on results from National Cooperative Highway Research Program Project 03-107 (*9*).

SPECIAL WORK ZONE CONFIGURATIONS

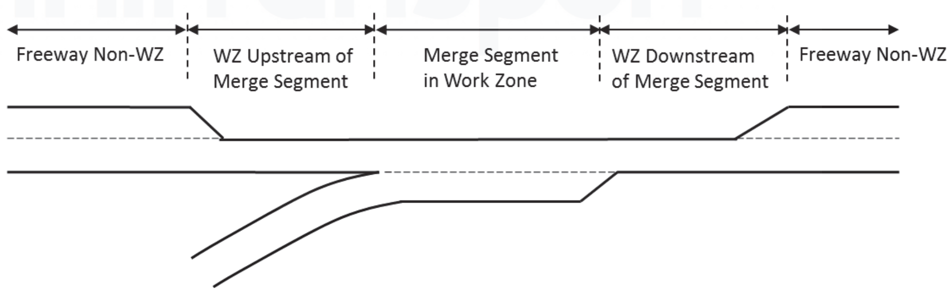
The queue discharge rate model predictions explained in Chapter 10 apply to basic freeway segments. These estimates should be adjusted for special freeway work zone configurations, such as merge segments, diverge segments, weaving segments, and work zones with directional crossovers. The relationships presented in this section were derived from field-calibrated microsimulation models for the special work zone configurations.

No data were available for the impacts of these work zone configurations on FFS, and so FFS estimates for these configurations should be used only when local data are not available. One exception is the FFS for a directional crossover, which should be estimated from the geometric design of the configuration, and is used as an input to the queue discharge rate estimation for that work zone configuration.

Work Zone Capacity Adjustments for Merge Segments

The proportion of work zone capacity (in reference to the basic work zone capacity calculated in Chapter 10) that is allocated to the mainline flow in a merge segment is presented separately for locations upstream and downstream of the special work zone activity segment. Exhibit 25-7 shows an example for a merge area within a construction zone.

Exhibit 25-7
On-Ramp Merge Diagram for 2-to-1 Freeway Work Zone Configuration



Note: WZ = work zone.

Exhibit 25-8 through Exhibit 25-12 give the proportion of work zone capacity allocated to mainline flow in merge, diverge, and directional crossover segments. For a weaving segment, a predictive model is presented following those exhibits. In the exhibits, only a subset of potential work zone configurations is presented, as these are the only ones that were included in the simulation modeling effort in the original research.

Exhibit 25-8 presents the proportion of available capacity upstream of a merge area in a construction zone, as a function of work zone lane configurations, different levels of on-ramp input volumes, and lengths of the acceleration lane. Upstream of the work zone, the proportion of capacity available to the mainline movement decreases considerably as the on-ramp demand increases.

Exhibit 25-8
Proportion of Work Zone Queue Discharge Rate (Relative to the Basic Work Zone Capacity) Available for Mainline Flow Upstream of Merge Area

Work Zone Lane Configuration	On-Ramp Input Demand (pc/h)	Acceleration Lane Length (ft)							
		100	300	500	700	900	1,100	1,300	1,500
2 to 1	0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	250	1.00	0.86	0.86	0.86	0.86	0.86	0.86	0.86
	500	1.00	0.70	0.70	0.70	0.70	0.70	0.70	0.70
	750	1.00	0.53	0.53	0.53	0.53	0.53	0.53	0.53
	1,000	1.00	0.49	0.45	0.40	0.40	0.40	0.40	0.40
2 to 2	0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	250	1.00	0.92	0.92	0.92	0.92	0.92	0.92	0.92
	500	1.00	0.84	0.84	0.84	0.84	0.84	0.84	0.84
	750	1.00	0.75	0.75	0.75	0.75	0.75	0.75	0.75
	1,000	1.00	0.67	0.67	0.67	0.67	0.67	0.67	0.67
3 to 2	0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	250	1.00	0.95	0.95	0.95	0.95	0.95	0.95	0.95
	500	1.00	0.87	0.87	0.87	0.87	0.87	0.86	0.86
	750	1.00	0.78	0.78	0.78	0.78	0.78	0.78	0.78
	1,000	1.00	0.70	0.70	0.70	0.70	0.70	0.70	0.70
4 to 3	0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	250	1.00	0.97	0.97	0.98	0.98	0.98	0.98	0.98
	500	1.00	0.91	0.91	0.91	0.92	0.92	0.92	0.92
	750	1.00	0.85	0.85	0.85	0.86	0.86	0.86	0.86
	1,000	1.00	0.79	0.79	0.79	0.79	0.80	0.80	0.80

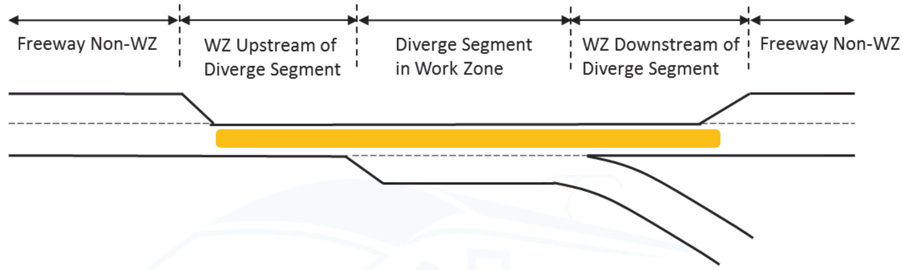
The capacity of the merge segment is the same as a basic work zone segment, with the caveat that the on-ramp flow consumes a portion of the mainline capacity. As a result, the available capacity upstream of the merge area leading into the work zone will be reduced once the queue spills back to the lane drop point. The proportions presented in Exhibit 25-8 approximate the conditions of a zipper merge configuration, with capacity divided approximately equally between the on-ramp and the right-most freeway mainline lane. In other words, the estimates correspond to a worst-case scenario for mainline flow in terms of available capacity, and a best-case scenario for the on-ramp movement. Note that the proportions for a 100-ft acceleration lane length are all 1.0 because on-ramp vehicles will experience difficulty entering the mainline lanes with the extremely short acceleration lane. These findings are based on results from microscopic simulation models of this configuration.

Research (*9*) shows that the throughput downstream of a merge area is approximately equal to the upstream queue discharge rate (before the merge) in most cases, with some configurations actually showing a marginal increase in flow. This slight increase occurs because additional demand from the on-ramp is able to more efficiently utilize gaps in the work zone queue discharge flow without the turbulence effects of the upstream lane drop. This effect was primarily observed for long acceleration lanes. However, for a more conservative estimate of work zone operations, it is recommended not to consider this increase in flow downstream of the merge area regardless of lane configuration, on-ramp input volume, or acceleration lane length.

Work Zone Capacity Adjustments for Diverge Segments

Similar to merge segment analysis, the analysis of diverge segments distinguishes between the diverge segment portions of the work zone that are upstream and downstream of the diverge segment. Exhibit 25-9 shows an example for a diverge area within a construction zone.

Exhibit 25-9
Off-Ramp Diverge Diagram for a 2-to-1 Freeway Work Zone Configuration



Note: WZ = work zone.

Exhibit 25-10 presents the proportion of available capacity downstream of a diverge area for various freeway work zone lane configurations, different levels of off-ramp volume percentage, and deceleration lane lengths. Upstream of the diverge area, research (*9*) shows the available capacity is generally equivalent to that of a basic work zone segment. Therefore, it is recommended to apply a fixed adjustment of 1.00 upstream of the diverge area regardless of lane configuration, off-ramp volume percentage, or deceleration lane length.

At the downstream end, however, the proportion of available capacity for mainline volume decreases significantly as the off-ramp volume percentage increases. Analysts should expect work zone operations to improve downstream of a diverge segment (but still within the work zone) because some portion of traffic will exit the freeway, thereby decreasing the processed volume below the downstream capacity. However, if the deceleration lane lengths are shorter than 100 ft, exiting vehicles will need to slow down while still on the mainline to complete the exit maneuver. This speed reduction may drop mainline capacity by as much as 10% or more.

For a diverge area, the proportion of off-ramp demand that can be served in the work zone under congested conditions can be predicted as presented in Exhibit 25-11. This proportion is defined as the off-ramp observed volume divided by the off-ramp demand volume.

Work Zone Lane Configuration	Off-Ramp Volume Percentage	Deceleration Lane Length (ft)							
		100	300	500	700	900	1,100	1,300	1,500
2 to 1	0.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	6.3	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.93
	12.5	0.87	0.88	0.88	0.88	0.88	0.88	0.87	0.87
	18.8	0.79	0.82	0.82	0.82	0.82	0.81	0.81	0.81
	25.0	0.72	0.76	0.76	0.75	0.75	0.75	0.75	0.75
2 to 2	0.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	6.3	0.93	0.94	0.94	0.94	0.94	0.94	0.94	0.94
	12.5	0.84	0.87	0.87	0.87	0.87	0.87	0.87	0.87
	18.8	0.76	0.81	0.81	0.81	0.81	0.81	0.81	0.81
	25.0	0.68	0.75	0.75	0.75	0.75	0.75	0.75	0.75
3 to 2	0.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	6.3	0.93	0.94	0.94	0.94	0.94	0.94	0.94	0.94
	12.5	0.86	0.87	0.87	0.87	0.87	0.87	0.87	0.87
	18.8	0.78	0.81	0.81	0.81	0.81	0.81	0.81	0.81
	25.0	0.69	0.74	0.74	0.74	0.74	0.74	0.74	0.74
4 to 3	0.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	6.3	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
	12.5	0.86	0.87	0.87	0.87	0.87	0.87	0.87	0.87
	18.8	0.76	0.80	0.80	0.80	0.80	0.80	0.80	0.80
	25.0	0.64	0.73	0.73	0.73	0.73	0.73	0.73	0.73

Exhibit 25-10
Proportion of Work Zone Capacity Available for Mainline Flow Downstream of Diverge Area

Lane Configuration	Proportion of Off-Ramp Demand Served in Work Zone
2 to 1	0.39
2 to 2	0.82
3 to 2	0.53
4 to 3	0.60

Exhibit 25-11
Proportion of Off-Ramp Demand Served in Work Zone

Work Zone Capacity Adjustments for Crossover Segments

Exhibit 25-12 presents the proportion of work zone capacity available for a directional crossover for various crossover vehicle speeds. As shown in the exhibit, the crossover capacity is highly sensitive to average crossover speed. The variation in capacity for different work zone lane configurations was found to be negligible in crossovers. The estimates in Exhibit 25-12 should be applied as multipliers of the basic segment work zone capacity described above.

Lane Configuration	Crossover Average Speed (mi/h)		
	25	35	45
2 to 1 3 to 2 4 to 3	0.83	0.90	0.94

Exhibit 25-12
Proportion of Available Work Zone Capacity for a Directional Crossover in the Work Zone

Work Zone Capacity Adjustments for Weaving Segments

In a weaving area, the proportion of work zone capacity available for mainline flow can be predicted by using a two-step model. In Step 1, the analyst estimates the maximum proportion of mainline flow that can be served through the work zone based on the work zone lane configuration and the volume ratio. This maximum becomes an upper bound on the actual estimated proportion, which is estimated in Step 2. In Step 2, the actual proportion of work zone capacity available for mainline flow is estimated based on the lane configuration, volume ratio, and auxiliary lane length. The final proportion of mainline flow that can be processed through the weaving segment is the lower of the two estimated proportions from Steps 1 and 2. The model intercept and coefficient values for Equation 25-37 and Equation 25-38 are presented in Exhibit 25-13.

Step 1: Estimate Maximum Mainline Allocation Proportion

Equation 25-37

$$MaxProportion = \text{Intercept} + \beta_1(\text{2-to-1}) + \beta_2(\text{2-to-2}) + \beta_3(\text{3-to-2}) + \beta_4(\text{4-to-3}) + \beta_5(VR)$$

where

$MaxProportion$ = maximum proportion of work zone capacity available for mainline flow at the weave area (decimal),

Intercept = model intercept,

β_1 = model coefficient for 2-to-1 lane closures,

2-to-1 = indicator variable that is 1 when the work zone has a 2-to-1 configuration and 0 otherwise,

β_2 = model coefficient for 2-to-2 lane closures,

2-to-2 = indicator variable that is 1 when the work zone has a 2-to-2 configuration and 0 otherwise,

β_3 = model coefficient for 3-to-2 lane closures,

3-to-2 = indicator variable that is 1 when the work zone has a 3-to-2 configuration and 0 otherwise,

β_4 = model coefficient for 4-to-3 lane closures,

4-to-3 = indicator variable that is 1 when the work zone has a 4-to-3 configuration and 0 otherwise,

β_5 = model coefficient for volume ratio, and

VR = volume ratio = weave volume/total volume.

Step 2: Predict Mainline Proportion

Equation 25-38

$$\text{Proportion} = \text{Intercept} + \beta_1(\text{2-to-1}) + \beta_2(\text{2-to-2}) + \beta_3(\text{3-to-2}) + \beta_4(\text{4-to-3}) + \beta_5(VR) + \beta_6(AuxLength)$$

where

Proportion = proportion of work zone capacity available for mainline flow (decimal),

β_6 = model coefficient for auxiliary lane length,

$AuxLength$ = auxiliary lane length (ft), and

all other variables are as defined previously.

The off-ramp demand volume proportion *Prop*(*off-ramp*) in the weaving area is estimated from Equation 25-39, with the intercept and model coefficients given in Exhibit 25-14, and all other variables as defined previously.

Equation 25-39

$$Prop(\textit{off-ramp}) = \text{Intercept} + \beta_1(\text{2-to-1}) + \beta_2(\text{2-to-2}) + \beta_3(\text{3-to-2}) + \beta_4(\text{4-to-3}) + \beta_5(VR)$$

Model		Model Term	Coefficient
Step 1: Maximum Proportion	Upstream	Intercept	1.0023
		β_1	–0.1197
		β_2	0.0105
		β_3	0.0085
		β_4	0.0000
		β_5	–0.3048
	Downstream	Intercept	1.0573
		β_1	0.1307
		β_2	–0.0623
		β_3	0.0494
		β_4	0.0000
		β_5	-0.3332
Step 2: Predicted Proportion	Upstream	Intercept	0.8491
		β_1	–0.0665
		β_2	0.0061
		β_3	0.0050
		β_4	0.0000
		β_5	–0.4687
		β_6	9.0956×10^{-5}
	Downstream	Intercept	0.8962
		β_1	0.2702
		β_2	0.0535
		β_3	0.1073
		β_4	0.0000
		β_5	–0.9694
		β_6	30.5253×10^{-5}

Exhibit 25-13
Model Coefficients for Estimating the Proportion of Work Zone Capacity in a Weaving Segment

Model	Model Term	Coefficient
Off-Ramp Volume Proportion	Intercept	0.6162
	β_1	–0.2201
	β_2	0.2082
	β_3	–0.0551
	β_4	0.0000
	β_5	0.0850

Exhibit 25-14
Model Coefficients for Estimating the Proportion of Off-Ramp Volume Served in the Weaving Area

6. PLANNING-LEVEL METHODOLOGY FOR FREEWAY FACILITIES

This section presents a planning-level approach for freeway facility analysis that is compatible with the operational method presented in Chapter 10, Freeway Facilities Core Methodology. The planning level-approach is specifically constructed to

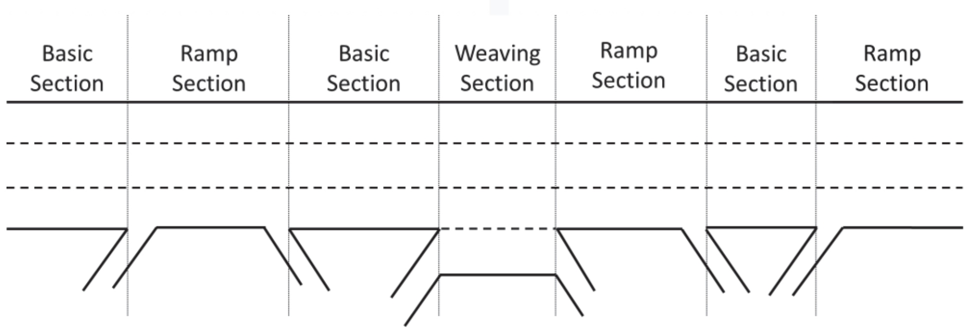
1. Use default values for as many of the operational parameters as practical;
2. Omit the need to enter detailed data about segment attributes (e.g., acceleration lane length and detailed weaving section geometry);
3. Aggregate the analysis to a coarser spatial representation, reporting at the freeway section level instead of the HCM segment level; and
4. Enable HCM users to manually carry out the analysis for a single peak hour without an extensive computational burden.

The method covers both undersaturated and oversaturated conditions and produces estimates of travel time, speed, density, and level of service (LOS). The underlying methodology relies on developing a relationship between *delay rate* per unit distance on a basic freeway segment, and the demand-to-capacity ratio. For weaving segments, capacity adjustment factors (CAFs) are developed based on the volume ratio and segment length. By using these factors, demand-to-capacity ratios on weaving segments can be adjusted, and the segment is subsequently treated similarly to a basic freeway segment. The capacities of merge and diverge segments are determined from the demand level, FFS, and space mean speed. CAFs are subsequently calculated for those segments, and their demand-to-capacity ratios are adjusted accordingly.

INPUT REQUIREMENTS

Input variables are characterized into global and section inputs. Sections are defined to occur between points where either demand or capacity changes, as shown in Exhibit 25-15.

Exhibit 25-15
Schematics of Freeway Sections



For instance, the first section in Exhibit 25-15 (starting from the left) is a basic freeway section. This section is followed by an on-ramp, and the demand level changes. Capacity and demand remain unchanged until the first off-ramp. Consequently, the second freeway section in Exhibit 25-15 is defined as a ramp section. The next section that follows is a basic freeway section. It is followed by a weaving section (this section is a weaving section due to the presence of an auxiliary lane). The weaving section is followed by another ramp section (due to an off-ramp), a basic section, and finally a ramp section (due to an on-ramp). Introduction of freeway sections facilitates user input and is more compatible with links in travel demand models as well as modern digital data sources.

In the operational freeway facilities method, the influence area of an on-ramp or off-ramp is typically limited to a length of 1,500 ft. In the planning method, ramp sections can be longer. For cases where a ramp section length exceeds 2 mi, it is recommended to divide the section into multiple sections to avoid having the lower ramp section capacity apply for a very long distance.

Global inputs include information about the facility of interest and are applicable to all sections across all analysis periods. These inputs include

1. Free-flow speed (S_{FFS}),
2. Peak hour factor (PHF),
3. Percentage heavy vehicles ($\%HV$),
4. General terrain type for truck passenger-car equivalent (PCE) conversion,
5. K-factor [to convert directional annual average daily traffic (AADT) to peak hour flows], and
6. Traffic growth factor (f_{tg}).

The equation used to estimate section speeds in this planning method (Equation 25-45) is fully consistent with the basic freeway segment speed–flow models presented in Chapter 12, Basic Freeway and Multilane Highway Segments. Section inputs cover information that is applicable to a given section across all analysis periods and that may vary from one section to another as a function of

1. Section type (basic, weave, ramp),
2. Section length L (mi),
3. Section number of lanes, and
4. Section directional AADT.

This information, along with the global inputs, is used to calculate the free-flow travel rate (the inverse of FFS), CAFs for weave and ramp sections, adjusted lane capacity (the product of base capacity and CAF), and section capacity (the product of adjusted lane capacity and number of lanes). The planning methodology follows five basic steps:

1. Demand-level calculations;
2. Section capacity calculations and adjustments;
3. Delay rate estimation;
4. Average travel time, speed, and density calculations; and
5. Level of service.

All steps are described in detail below.

STEP 1: DEMAND-LEVEL CALCULATIONS

The demand level for each section is determined from the entering demand, exiting demand, and carryover demand from a previous analysis period (in the case of oversaturated conditions).

The methodology uses the directional average annual daily traffic on section i $AADT_i$, K-factor, traffic growth factor f_{tg}, and peak hour factor PHF during each 15-min analysis period t in the peak hour to compute the demand inflow and outflow $V_{i,t}$ as shown in Equation 25-40:

Equation 25-40

$$V_{i,t} = \begin{cases} AADT_i \times k \times f_{tg} & t = 1, 3 \\ AADT_i \times k \times \left(\frac{1}{PHF}\right) \times f_{tg} & t = 2 \\ AADT_i \times k \times \left(2 - \frac{1}{PHF}\right) \times f_{tg} & t = 4 \end{cases}$$

where all parameters were defined previously.

All demand inputs should be in units of passenger cars per hour per lane (pc/h/ln). If demands are given in units of vehicles per hour per lane (veh/h/ln), they need to be converted with Equation 25-41.

Equation 25-41

$$q_{i,t} = \frac{V_{i,t}}{f_{HV}}$$

where

$q_{i,t}$ = demand flow rate in PCEs (pc/h),

$V_{i,t}$ = demand flow rate in vehicles per hour (veh/h), and

f_{HV} = adjustment factor for presence of heavy vehicles in traffic stream.

Just as in the operational method, all heavy vehicles are classified as single-unit trucks (SUTs) or tractor-trailers (TTs). Recreational vehicles and buses are treated as SUTs. The heavy-vehicle adjustment factor f_{HV} is computed from the combination of the two heavy vehicle classes, which are added to get an overall truck percentage P_T, as shown by Equation 25-42.

Equation 25-42

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)}$$

where

f_{HV} = heavy-vehicle adjustment factor (decimal),

P_T = proportion of SUT and TTs in traffic stream (decimal), and

E_T = PCE of one heavy vehicle in the traffic stream (PCE).

The values for E_T are 2.0 for level terrain and 3.0 for rolling terrain. For specific grades, Chapter 12 provides other heavy-vehicle equivalency factors.

The converted demand flow rates $q_{i,t}$ can represent both inflow demand and outflow demand. For the first facility section and all on-ramps, $q_{i,t}$ represents inflow demand and is denoted by $(q_{i,z})_{\text{in}}$. For all off-ramps, $q_{i,t}$ represents outflow demand and is represented by $(q_{i,z})_{\text{out}}$.

Demand level $d_{i,t}$ (in passenger cars per hour) on section i in analysis period t is computed as the demand level in section $i - 1$, plus the inflow at section i during analysis period t, minus the outflow at the same section at analysis period t, plus any carryover demand $d'_{i,t-1}$ in section i from the previous analysis period $t - 1$. The relationship is as shown in Equation 25-43.

Equation 25-43

$$d_{i,t} = d_{i-1,t} + \left(q_{i,t}\right)_{\text{in}} - \left(q_{i,t}\right)_{\text{out}} + d'_{i,t-1}$$

where all variables are as defined previously.

The carryover demand $d'_{i,t-1}$ on section i at analysis period t is the difference between the section demand and capacity, as given by Equation 25-44.

Equation 25-44

$$d'_{i,t} = \max\left(d_{i,t} - c_i, 0\right)$$

The carryover demand is also used as an indication of a queue on the section. Note that in this approach, queues are stacked vertically and do not spill back into an upstream link. The section queue length is estimated by dividing the difference in lane demand and capacity by the density. Essentially, it provides an estimate for how long the queue would spill back at the given density, assuming a fixed number of lanes upstream of the bottleneck.

STEP 2: SECTION CAPACITY CALCULATIONS AND ADJUSTMENTS

The capacity of basic freeway sections is found by using the FFS and the percentage of heavy vehicles on the facility, as shown by Equation 25-45.

Equation 25-45

$$c_i = 2{,}200 + 10 \times [\min(70, S_{FFS}) - 50]$$

where c_i is the capacity of freeway section i (pc/h/ln) and S_{FFS} is the facility's free-flow speed (mi/h).

Equation 25-45 provides capacity values for basic freeway sections. This capacity must be adjusted for weaving, merge, diverge, and ramp sections, as described next.

Capacity Adjustments for Weaving Sections

As mentioned above, the planning method is derived from the basic freeway segment speed–flow model to estimate a section's delay rate and travel speed. When applied to weaving sections, an adjustment to capacity is required to account for the generally lower capacity in weaving segments. This capacity adjustment factor CAF_{weave} can be estimated with Equation 25-46.

Equation 25-46

$$CAF_{\text{weave}} = \min(0.884 - 0.0752V_r + 0.0000243L_s, 1)$$

where

CAF_{weave} = capacity adjustment factor used for a weaving segment ($0 \le CAF_{\text{weave}} \le 1.0$) (decimal),

V_r = ratio of weaving demand flow rate to total demand flow rate in the weaving segment (decimal), and

L_s = weaving segment length (mi).

Through this capacity adjustment, the basic section method can be extended to weaving sections, as described elsewhere (*10*). The process for estimating

CAF_{weave} is based on a representative weaving section with the following characteristics (see Chapter 13 for additional details):

- Minimum number of lane changes that must be made by a single weaving vehicle from the on-ramp to the freeway: $LC(RF) = 1$,
- Minimum number of lane changes that must be made by a single weaving vehicle from the freeway to the off-ramp: $LC(FR) = 1$,
- Minimum number of lane changes that must be made by a ramp-to-ramp vehicle to complete a weaving maneuver: $LC(RR) = 0$, and
- Number of lanes from which a weaving maneuver may be made with one or no lane changes: $N(WL) = 2$.

Adjustments for Ramp Sections

Research shows an average CAF of 0.9 can be used for ramp sections with an on-ramp or off-ramp (*10*, *11*). It is recognized that known bottlenecks may have significantly reduced capacities that require a lower CAF. Further calibration of the CAF by the analyst is strongly encouraged when applying this method to on-ramp sections with known capacity constraints and congestion impacts. Analyst calibration of this factor is also possible for off-ramp sections.

STEP 3: DELAY RATE ESTIMATION

The planning-level approach estimates the delay rate per unit distance as a function of a section's demand-to-capacity ratio. The delay rate is the difference between the actual and free-flow travel time per unit distance. For example, if a facility's space mean speed is 60 mi/h relative to an FFS of 75 mi/h for a 0.5-mi segment, then the free-flow travel time is 0.4 min, and the actual travel time is 0.5 min. The delay rate per mile is the difference of those travel times divided by the segment length, which gives a delay rate of 0.2 min/mi. The calculation of the delay rate needs to be performed differently for undersaturated and oversaturated conditions, as described next.

Undersaturated Conditions

For undersaturated conditions, the basic freeway segment speed–flow model in Chapter 12 can be used to estimate delay rates. However, for a planning-level analysis, it is desirable to further simplify the estimation of delay rate to be a function of inputs readily available in a planning context. The delay rate $\Delta_{RU_{i,t}}$ (in minutes per mile) for segment i in time period t as a function of the demand-to-capacity ratio $d_{i,t}/c_i$ is given by Equation 25-47.

Equation 25-47

$$\Delta_{RU_{i,t}} = \begin{cases} 0 & \dfrac{d_{i,t}}{c_i} < E \\ A\left(\dfrac{d_{i,t}}{c_i}\right)^3 + B\left(\dfrac{d_{i,t}}{c_i}\right)^2 + C\left(\dfrac{d_{i,t}}{c_i}\right) + D & E \le \dfrac{d_{i,t}}{c_i} \le 1 \end{cases}$$

where A, B, C, D, and E are parameters given in Exhibit 25-16 and all other variables are as defined previously.

Free-Flow Speed (mi/h)	A	B	C	D	E
75	68.99	–77.97	34.04	–5.82	0.44
70	71.24	–85.48	35.58	–5.44	0.52
65	92.45	–127.33	56.34	–8.00	0.62
60	121.35	–184.84	83.21	–9.33	0.72
55	156.43	–248.99	99.20	–0.12	0.82

Exhibit 25-16 Parameter Values for Undersaturated Model

Oversaturated Conditions

For oversaturated conditions, the additional delay rate is approximated assuming uniform arrival and departures at the bottleneck location. With the demand exceeding capacity, any demand that cannot be served through the bottleneck must be stored upstream of the bottleneck in a queue. The *additional* oversaturation delay rate $\Delta_{RO_{i,t}}$ (in minutes per mile) for segment i at analysis period t, over a 15-min (900-s) analysis period, is obtained by Equation 25-48.

$$\Delta_{RO_{i,t}} = \frac{450}{L}\left[\max\left(\frac{d_{i,t}}{c_i} - 1.0\right)\right]$$

Equation 25-48

where all variables are as previously defined.

STEP 4: AVERAGE TRAVEL TIME, SPEED, AND DENSITY CALCULATIONS

After the delay rate is determined, the travel rate is computed by summing the delay rate and travel rate under free-flow conditions, as shown by Equation 25-49.

$$TR_{i,t} = \Delta_{RU_{i,t}} + \Delta_{RO_{i,t}} + TR_{FFS}$$

Equation 25-49

where $TR_{i,t}$ is the travel rate on segment i in analysis period t (min/mi), TR_{FFS} is the travel rate under free-flow conditions (min/mi), and all other parameters are as previously defined.

The section travel time is then computed by multiplying the travel rate and segment length, as shown by Equation 25-50.

$$T_{i,t} = TR_{i,t} \times L_i$$

Equation 25-50

where $T_{i,t}$ is the travel time on segment i in analysis period t (min/mi), $TR_{i,t}$ is the travel rate on segment i in analysis period t (min/mi), and L_i is the length of section i (mi).

The average speed $S_{i,t}$ (in miles per hour) on section i in analysis period t is computed by using Equation 25-51.

$$S_{i,t} = \frac{L_i}{T_{i,t}}$$

Equation 25-51

Finally, the density is calculated as shown by Equation 25-52.

$$D_{i,t} = \frac{d_{i,t}}{S_{i,t}}$$

Equation 25-52

where $D_{i,t}$ is density on section i in analysis period t (pc/mi), $d_{i,t}$ is section demand (pc/h), and $S_{i,t}$ is speed (mi/h).

Thus, the planning-level method provides a facility performance summary that includes whether the facility is undersaturated or oversaturated, the total facility travel time, the space mean speed, the average facility density, and the total queue length.

STEP 5: LEVEL OF SERVICE

With the density obtained in Step 4, LOS can be estimated for urban or rural facilities following the thresholds in Chapter 10.

The LOS criteria for urban and rural freeway facilities are repeated in Exhibit 25-17. Urban LOS thresholds are the same density-based criteria used for basic freeway segments. Studies on LOS perception by rural travelers indicate lower-density thresholds than those of their urban freeway counterparts. The average LOS applies to each 15-min time period.

Exhibit 25-17
LOS Criteria for Urban and Rural Freeway Facilities

LOS	Urban Freeway Facility Density (pc/mi/ln)	Rural Freeway Facility Density (pc/mi/ln)
A	≤11	≤6
B	>11–18	>6–14
C	>18–26	>14–22
D	>26–35	>22–29
E	>35–45	>29–39
F	>45 or any component section v_d/c ratio > 1.00	>39 or any component section v_d/c ratio >1.00

7. MIXED-FLOW MODEL FOR COMPOSITE GRADES

This section presents the application of the mixed-flow model in the case of composite grades. The procedure builds on the single-grade methodology described in Chapter 26, Freeway and Highway Segments: Supplemental, and uses the same basic set of equations. The procedure computes LOS, capacity, speed, and density for each segment and for the composite grade as a whole. Many of the equations in this section are identical to those presented in Chapter 26, although they have different equation numbers. The major difference with composite grades is that the analyst must compute the spot travel rates or spot speeds at the start and end of each segment on the composite grade as an input to the analysis of the next grade segment.

OVERVIEW OF THE METHODOLOGY

The methodology assumes the composite grade both begins and ends with a long, level segment. The example shown in Exhibit 25-18 has five segments.

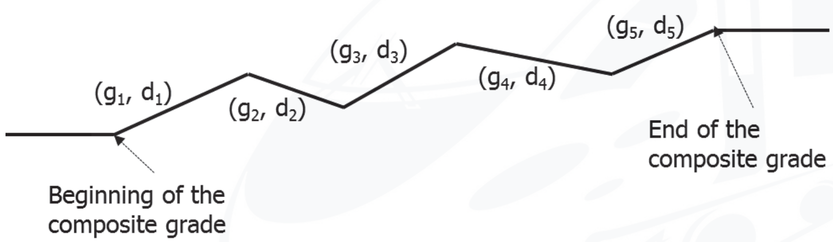


Exhibit 25-18
Schematic of a Composite Grade

Exhibit 25-19 presents the methodology flowchart. The remainder of this section provides the computational details for each step in the process.

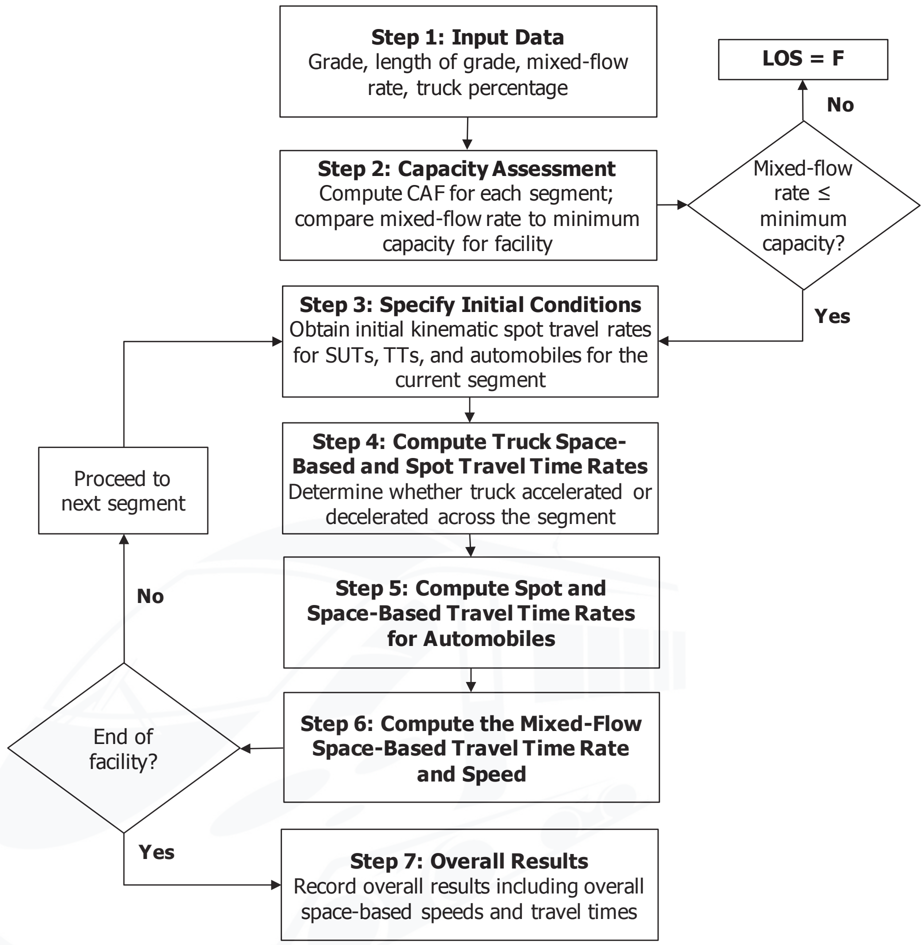
STEP 1: INPUT DATA

The user must supply the length d_j (mi) and the grade g_j (decimal) for each segment j, including the tangent segment immediately preceding the composite grade. In addition, the auto-only free-flow speed FFS (mi/h), peak hour factor PHF (decimal), the flow rate of mixed traffic v_{mix} (veh/h/ln), and the fraction of SUTs and TTs in the traffic stream must be specified for the facility as a whole.

STEP 2: CAPACITY ASSESSMENT

Before the composite grade is examined in detail, the capacity of the individual segments j is determined. A mixed-flow capacity adjustment factor $CAF_{mix,j}$ converts auto-only capacities into mixed-traffic-stream capacities. It is computed with Equation 25-53. The third term in this equation changes for each segment.

Exhibit 25-19
Mixed-Flow Methodology Overview



Equation 25-53

$$CAF_{\text{mix},j} = CAF_{ao} - CAF_{T,\text{mix}} - CAF_{g,\text{mix},j}$$

where

$CAF_{\text{mix},j}$ = mixed-flow capacity adjustment factor for segment j (decimal),

CAF_{ao} = capacity adjustment factor for the auto-only case (e.g., due to weather or incidents) (decimal),

$CAF_{T,\text{mix}}$ = capacity adjustment factor for the percentage of trucks in mixed-flow conditions (decimal), and

$CAF_{g,\text{mix},j}$ = capacity adjustment factor for grade for segment j in mixed-flow conditions (decimal).

CAF for the Auto-Only Case

Because CAF_{ao} is used to convert auto-only capacities into mixed-traffic capacities, it defaults to a value of 1.0 unless other capacity adjustments are in effect (e.g., weather, incidents, driver population factor).

CAF for Truck Percentage

The CAF for truck percentage $CAF_{T,\mathrm{mix}}$ is computed with Equation 25-54.

$$CAF_{T,\mathrm{mix}} = 0.53 \times {P_T}^{0.72}$$

Equation 25-54

where P_T is the total percentage of SUTs and TTs in the traffic stream (decimal).

CAF for Grade Effect

The CAF for grade effect $CAF_{g,\mathrm{mix}}$ accounts for the grade severity, grade length, and truck presence. It is computed by using Equation 25-55 with Equation 25-56.

$$\begin{aligned} CAF_{g,\mathrm{mix}} = \rho_{g,\mathrm{mix}} &\times \max[0, 0.69 \times (e^{12.9 g_j} - 1)] \\ &\times \max[0, 1.72 \times (1 - 1.71 e^{-3.16 d_j})] \end{aligned}$$

Equation 25-55

with

$$\rho_{g,\mathrm{mix}} = \begin{cases} 8 \times P_T & P_T < 0.01 \\ 0.126 - 0.03 P_T & \text{otherwise} \end{cases}$$

Equation 25-56

where

$\rho_{g,\mathrm{mix}}$ = coefficient for grade term in the mixed-flow CAF equation (decimal),

P_T = total truck percentage (decimal),

g_j = grade of segment j (decimal), and

d_j = length of segment j (mi).

Once $CAF_{\mathrm{mix},j}$ is computed, the mixed-flow capacity for each segment j is calculated with Equation 25-57.

$$C_{\mathrm{mix},j} = C_{ao} \times CAF_{\mathrm{mix},j}$$

Equation 25-57

where

$C_{\mathrm{mix},j}$ = mixed-flow capacity for segment j (veh/h/ln);

C_{ao} = auto-only capacity for the given FFS, from Exhibit 12-6 (pc/h/ln); and

$CAF_{\mathrm{mix},j}$ = mixed-flow capacity adjustment factor for segment j (decimal).

The procedure identifies the smallest of these capacities and designates it as C_{mix}. It also notes the segment that produces this capacity as j_c. The capacity C_{mix} is checked against the mixed-flow rate v_{mix} to check if $v_{\mathrm{mix}} \geq C_{\mathrm{mix}}$. If this condition occurs, the system is deemed to be oversaturated, LOS F is reported, and no further analysis is carried out. However, if $v_{\mathrm{mix}} < C_{\mathrm{mix}}$, the procedure continues.

STEP 3: SPECIFY INITIAL CONDITIONS

Starting with Step 3, the methodology analyzes each segment in sequence. Steps 3 through 6 are repeated for each segment until the final segment on the composite grade is reached. The main focus is on computing travel times and speeds for SUTs, TTs, and autos.

Step 3 specifies the initial kinematics-based spot speeds for SUTs and TTs. The effects of the traffic interaction terms are omitted for the time being. The focus is on the kinematic behavior of the trucks as they ascend and descend the individual grades. For the first segment, the initial kinematic spot speed is the speed for SUTs and TTs on the long, level segment that precedes the composite grade. For all subsequent segments, it is the kinematic spot speed at the end of the previous segment. The kinematic spot speeds are speeds without traffic interaction, which will be added to the final kinematic spot speeds to obtain final spot speeds of each segment.

STEP 4: COMPUTE TRUCK SPOT AND SPACE-BASED TRAVEL TIME RATES

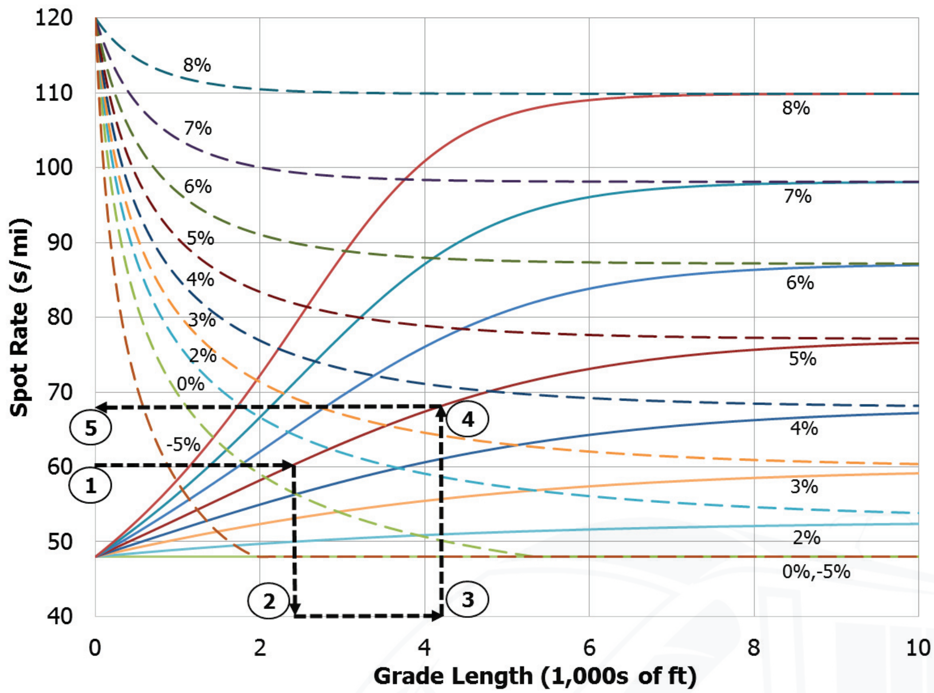
This step computes the SUT and TT space-based travel time rates for each of the segments and the spot rates at the end of each segment. The procedure follows a process similar to Step 5 of the mixed-flow model procedure described in Chapter 26.

The first substep involves analyzing the kinematic behavior of the trucks on the grade. The final spot rates are needed, as well as a determination of whether the trucks accelerated or decelerated on the grade.

Exhibit 25-20 and Exhibit 25-21 can be used for these purposes. These graphs are based on kinematic relationships given elsewhere (*12*). Alternative models of propulsive and resistive forces, such as more complex ones that account for gear shifting (e.g., *13*, *14*), can produce longer travel times. Such considerations can be incorporated into the mixed-flow model by adjusting the parameter values that affect the tractive effort to account for the additional losses. The travel time rates presented here are based on a model that assumes constant peak-engine power. Other models (e.g., *13*, *14*) account for the power losses that occur for the time intervals prior to and after gear shifting when the engine speed is outside the range that produces peak power.

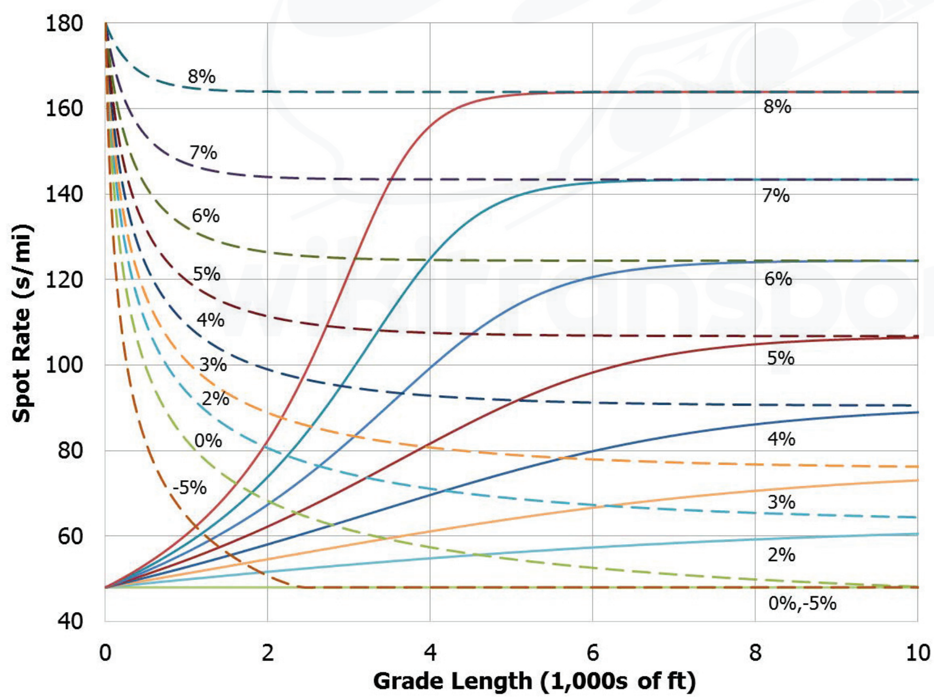
Exhibit 25-20 shows the trends in SUT spot rates for various grades starting from travel rates of 48 s/mi (75 mi/h) and 120 s/mi (30 mi/h). Exhibit 25-21 shows the same trends for a TT. Clearly, trucks decelerate as upgrades become steeper. For milder grades, trucks can often accelerate.

Exhibit 25-20
SUT Spot Rates Versus Distance with Initial Speeds of 75 and 30 mi/h



Notes: Curves in this graph assume a weight-to-horsepower ratio of 100. Solid curves are for an initial speed of 75 mi/h (48 s/mi) and dashed curves are for an initial speed of 30 mi/h (120 s/mi).

Exhibit 25-21
TT Spot Rates Versus Distance with Initial Speeds of 75 and 20 mi/h



Notes: Curves in this graph assume a weight-to-horsepower ratio of 100. Solid curves are for an initial speed of 75 mi/h (48 s/mi) and dashed curves are for an initial speed of 20 mi/h (180 s/mi).

In both Exhibit 25-20 and Exhibit 25-21, the *x*-axis gives the distance *d* traveled by the truck, and the *y*-axis gives the spot travel rate $\tau_{kin,j}$ at the end of that distance. The different curves are for various upgrades and downgrades.

To ascertain whether trucks accelerate or decelerate on segment *j*, consider the travel time rate trends shown in Exhibit 25-20 and Exhibit 25-21. If an SUT's final spot rate for segment *j* $\tau_{SUT,kin,f,j}$ is greater than the SUT's initial spot rate for segment *j* $\tau_{SUT,kin,i,j}$ and the TT's spot rate at the end of segment *j* $\tau_{TT,kin,f,j}$ is greater than the TT's spot rate at the beginning of segment *j* $\tau_{TT,kin,i,j}$, then both truck classes decelerate. If $\tau_{SUT,kin,f,j} < \tau_{SUT,kin,i,j}$ and $\tau_{TT,kin,f,j} < \tau_{TT,kin,i,j}$, then both truck classes accelerate.

To determine the end-of-grade spot travel time rates, start by finding the point on the applicable grade that corresponds to the initial kinematic rate. Treat that point as the zero distance location. Next, proceed along the grade length (*x*-axis) for a distance equal to the length *d* of the segment and read the spot rate at that distance. This reading is the final spot rate. For example, an SUT travels 2,000 ft starting from 60 mi/h (60 s/mi) on a 5% grade. Point 1 in Exhibit 25-20 is the 60-mi/h speed (60-s/mi rate) from which the SUT starts to travel on the 5% grade. Point 2 is the distance that is treated as the zero distance of the SUT. Point 3 represents the distance the SUT has traveled after 2,000 ft. The final spot rate can be read at Point 4. The initial kinematic SUT and TT spot rates for segment *j* $\tau_{SUT,kin,i,j}$ and $\tau_{TT,kin,i,j}$ are the kinematic spot rates at the end of the preceding segment. For remaining segments, $\tau_{SUT,kin,i,j}$ and $\tau_{TT,kin,i,j}$ are the kinematic spot rates at the end of the preceding segment $j - 1$, which are $\tau_{SUT,kin,f,j-1}$ and $\tau_{TT,kin,f,j-1}$.

The second substep involves determining the space-based travel time rates for SUTs and TTs. Exhibit 25-22 and Exhibit 25-23 provide examples. Exhibit 25-22 shows the time versus distance relationships for SUTs starting at 70 mi/h with a desired speed of 75 mi/h as they accelerate or decelerate on various grades. Exhibit 25-23 shows time versus distance relationships for SUTs starting at 30 mi/h as they ascend or descend grades. Relationships for a range of initial rates for both SUTs and TTs are provided in Appendix A.

In all exhibits, the *x*-axis is the distance *d* traveled by the truck, while the *y*-axis is the travel time *T* to cover the grade length *d*. The various curves in each exhibit represent different upgrades. All the truck profiles have a desired speed of 75 mi/h. For example, the 2% curve in Exhibit 25-23 shows travel time versus distance for SUTs starting from 30 mi/h with a desired speed of 75 mi/h.

When necessary, symbols are placed on the curves to indicate where a truck reaches 55, 60, 65, and 70 mi/h, for use when the speed limit is less than 75 mi/h, as indicated in the notes for Exhibit 25-23. For example, if the speed limit is 55 mi/h, it is assumed trucks will maintain a constant speed of 55 mi/h after reaching that speed. The analyst would use the graph to determine the travel time to accelerate to 55 mi/h and then perform the remainder of the travel time calculation using 55 mi/h as the truck speed. Not all curves have these symbols, as (*a*) the truck's crawl speed would be less than 55 mi/h for the particular grade, (*b*) the truck would take more than 10,000 ft to reach that speed, or (*c*) the graph being used starts from a relatively high speed (e.g., Exhibit 25-22).

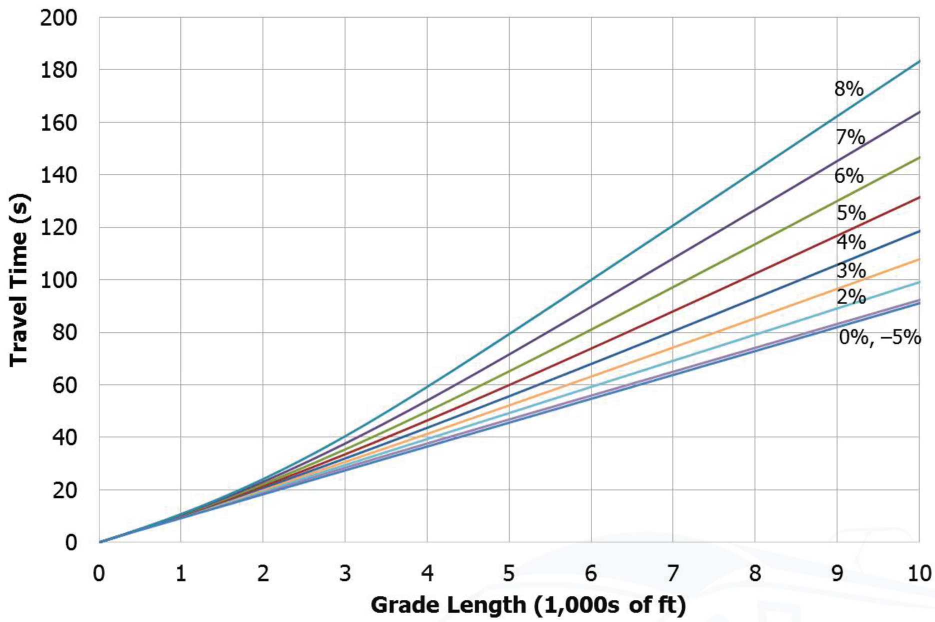


Exhibit 25-22
SUT Travel Time Versus Distance Curves for 70-mi/h Initial Speed

Note: Curves in this graph assume a weight-to-horsepower ratio of 100.

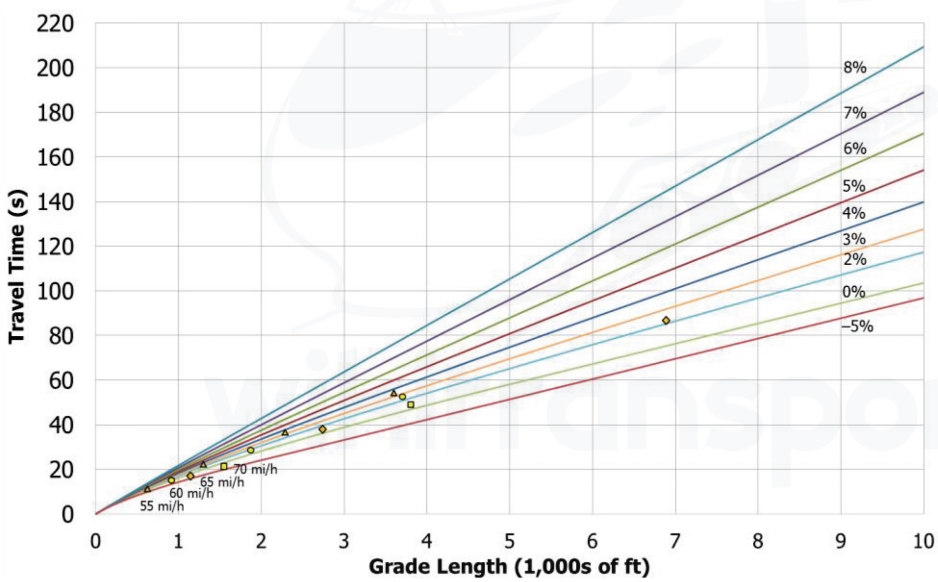


Exhibit 25-23
SUT Travel Time Versus Distance Curves for 30-mi/h Initial Speed

Notes: Curves in this graph assume a weight-to-horsepower ratio of 100.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

The analyst should use the Appendix A graph that has a starting spot speed closest to the value computed in the first substep. Because the graphs are provided in 5-mi/h increments, this choice means using the graph that is within 2.5 mi/h of the speed corresponding to the segment's initial spot rate.

The kinematic space-based travel time rate τ_{kin} (in seconds per mile) can then be computed with Equation 25-58.

$$\tau_{kin} = T/d$$

Equation 25-58

where T is the segment travel time (s) and d is the grade length (mi).

The maximum grade length shown in the graphs is 10,000 ft. When the grade length exceeds 10,000 ft, the travel rate can be computed using Equation 25-59.

Equation 25-59

$$\tau_{kin} = \frac{T_{10000}}{d} + \delta\left(1 - \frac{10{,}000}{5{,}280d}\right) \times 5{,}280$$

where

τ_{kin} = kinematic travel rate (s/mi),

T_{10000} = travel time at 10,000 ft (s),

δ = slope of the travel time versus distance curve (s/ft),

d = grade length (mi), and

5,280 = number of feet in 1 mi.

The δ values for SUTs and TTs are shown in Exhibit 25-24 and Exhibit 25-25, respectively.

Exhibit 25-24
δ Values for SUTs

	Free-Flow Speed (mi/h)					
Grade	**50**	**55**	**60**	**65**	**70**	**75**
–5%	0.0136	0.0124	0.0114	0.0105	0.0097	0.0091
0%	0.0136	0.0124	0.0114	0.0105	0.0097	0.0091
2%	0.0136	0.0124	0.0114	0.0105	0.0100	0.0099
3%	0.0136	0.0124	0.0114	0.0113	0.0112	0.0112
4%	0.0136	0.0129	0.0128	0.0128	0.0128	0.0127
5%	0.0146	0.0146	0.0146	0.0146	0.0145	0.0145
6%	0.0165	0.0165	0.0165	0.0165	0.0165	0.0165
7%	0.0186	0.0186	0.0186	0.0186	0.0186	0.0186
8%	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208

Exhibit 25-25
δ Values for TTs

	Free-Flow Speed (mi/h)					
Grade	**50**	**55**	**60**	**65**	**70**	**75**
–5%	0.0136	0.0124	0.0114	0.0105	0.0097	0.0091
0%	0.0136	0.0124	0.0114	0.0105	0.0097	0.0091
2%	0.0136	0.0124	0.0119	0.0118	0.0116	0.0115
3%	0.0143	0.0143	0.0142	0.0141	0.0140	0.0138
4%	0.0171	0.0171	0.0171	0.0170	0.0169	0.0168
5%	0.0202	0.0202	0.0202	0.0202	0.0202	0.0202
6%	0.0236	0.0236	0.0236	0.0236	0.0236	0.0236
7%	0.0272	0.0272	0.0272	0.0272	0.0272	0.0272
8%	0.0310	0.0310	0.0310	0.0310	0.0310	0.0310

Once the end-of-grade spot travel time rates and the space-based rates are obtained for the current segment, Equation 25-60 and Equation 25-61 are used to account for the traffic interaction term to obtain the actual truck spot and space-based travel time rates.

Equation 25-60

$$\tau_{*,SUT,j} = \tau_{*,SUT,kin,j} + \Delta\tau_{TI}$$

Equation 25-61

$$\tau_{*,TT,j} = \tau_{*,TT,kin,j} + \Delta\tau_{TI}$$

where

* = placeholder that can either be f to designate the spot travel time rate at the end of the segment or S to indicate the space-based rate across the segment,

$\tau_{*,SUT,j}$ = spot travel time rate for SUTs at the end of segment j or the space-based rate (s/mi),

$\tau_{*,SUT,kin,j}$ = kinematic final spot travel time rate or space-based rate for SUTs (s/mi),

$\Delta\tau_{TI}$ = traffic interaction term (s/mi) from Equation 25-62,

$\tau_{*,TT,j}$ = spot travel time rate for TTs at the end of segment j or the space-based rate (s/mi), and

$\tau_{*,TT,kin,j}$ = kinematic final spot travel time rate or space-based rate for TTs (s/mi).

The traffic interaction term represents the contribution of other traffic to truck speeds or travel time rates in mixed flow. It is computed by Equation 25-62.

$$\Delta\tau_{TI} = \left(\frac{3{,}600}{S_{ao}} - \frac{3{,}600}{FFS}\right) \times \left[1 + 3\left(\frac{1}{CAF_{\text{mix}}} - 1\right)\right]$$

Equation 25-62

where

$\Delta\tau_{TI}$ = traffic interaction term (s/mi),

S_{ao} = auto-only speed for the given flow rate (mi/h) from Equation 25-63,

FFS = base free-flow speed of the basic freeway segment (mi/h), and

CAF_{mix} = mixed-flow capacity adjustment factor for the segment (decimal) from Equation 25-53.

The auto-only travel time rate for the given flow rate can be computed with Equation 25-63.

$$S_{ao} = \begin{cases} FFS & \dfrac{v_{\text{mix}}}{CAF_{\text{mix}}} \le BP_{ao} \\ FFS - \dfrac{\left(FFS - \frac{C_{ao}}{D_c}\right)\left(\frac{v_{\text{mix}}}{CAF_{\text{mix}}} - BP_{ao}\right)^2}{(C_{ao} - BP_{ao})^2} & \dfrac{v_{\text{mix}}}{CAF_{\text{mix}}} > BP_{ao} \end{cases}$$

Equation 25-63

where

S_{ao} = auto-only speed for the given flow rate (mi/h),

FFS = base free-flow speed of the basic freeway segment (mi/h),

C_{ao} = base segment capacity (pc/h/ln) from Exhibit 12-6,

BP_{ao} = breakpoint in the auto-only flow condition (pc/h/ln) from Exhibit 12-6,

D_c = density at capacity = 45 pc/mi/ln,

v_{mix} = flow rate of mixed traffic (veh/h/ln), and

CAF_{mix} = mixed-flow capacity adjustment factor for the basic freeway segment (decimal).

STEP 5: COMPUTE AUTOMOBILE SPOT AND SPACE-BASED TRAVEL TIME RATES

Whether trucks accelerate or decelerate, the automobile spot travel time rates at the end of the segment are computed with Equation 25-64. The analyst should check that the automobile spot rates are always less than or equal to the truck spot rates (i.e., automobile speeds are greater than or equal to truck speeds).

Equation 25-64

$$\tau_{f,a,j} = \frac{3{,}600}{FFS} + \Delta\tau_{TI} + \left[\begin{array}{c} 64.50 \times \left(\frac{v_{\text{mix}}}{1{,}000}\right)^{0.77} \times (P_{SUT})^{0.34} \\ \times \max\left(0, \frac{\tau_{f,SUT,kin,j}}{100} - \frac{3{,}600}{FFS \times 100}\right)^{1.53} \end{array} \right] + \left[\begin{array}{c} 79.50 \times \left(\frac{v_{\text{mix}}}{1{,}000}\right)^{0.81} \times (P_{TT})^{0.56} \\ \times \max\left(0, \frac{\tau_{f,TT,kin,j}}{100} - \frac{3{,}600}{FFS \times 100}\right)^{1.32} \end{array} \right]$$

where

$\tau_{f,a,j}$ = end-of-grade spot travel time rate for automobiles (s/mi),

$\tau_{f,SUT,kin,j}$ = spot kinematic travel time rate of SUTs at the end of segment j (s/mi),

$\tau_{f,TT,kin,j}$ = spot kinematic travel time rate of TTs at the end of segment j (s/mi),

$\Delta\tau_{TI}$ = traffic interaction term (s/mi),

v_{mix} = flow rate of mixed traffic (veh/h/ln),

FFS = base free-flow speed of the basic freeway segment (mi/h),

P_{SUT} = proportion of SUTs in the traffic stream (decimal), and

P_{TT} = proportion of TTs in the traffic stream (decimal).

In Step 4, it was determined whether trucks accelerate or decelerate across a segment. If they decelerate, Equation 25-65 is used to compute the auto space-based travel time rate. If trucks accelerate, Equation 25-66 is employed. The auto space mean rates are always less than or equal to the truck space mean rates.

Equation 25-65

$$\tau_{S,a,j} = \frac{3{,}600}{FFS} + \Delta\tau_{TI} + \left[\begin{array}{c} 100.42 \times \left(\frac{v_{\text{mix}}}{1{,}000}\right)^{0.46} \times (P_{SUT})^{0.68} \\ \times \max\left(0, \frac{\tau_{S,SUT,kin,j}}{100} - \frac{3{,}600}{FFS \times 100}\right)^{2.76} \end{array} \right] + \left[\begin{array}{c} 110.64 \times \left(\frac{v_{\text{mix}}}{1{,}000}\right)^{1.36} \times (P_{TT})^{0.62} \\ \times \max\left(0, \frac{\tau_{S,TT,kin,j}}{100} - \frac{3{,}600}{FFS \times 100}\right)^{1.81} \end{array} \right]$$

$$\tau_{S,a,j} = \frac{3{,}600}{FFS} + \Delta\tau_{TI} + \begin{bmatrix} 54.72 \times \left(\frac{v_{\text{mix}}}{1{,}000}\right)^{1.16} \times (P_{SUT})^{0.28} \\ \times \max\left(0, \frac{\tau_{S,SUT,kin,j}}{100} - \frac{3{,}600}{FFS \times 100}\right)^{1.73} \end{bmatrix} + \begin{bmatrix} 69.72 \times \left(\frac{v_{\text{mix}}}{1{,}000}\right)^{1.32} \times (P_{TT})^{0.61} \\ \times \max\left(0, \frac{\tau_{S,TT,kin,j}}{100} - \frac{3{,}600}{FFS \times 100}\right)^{1.33} \end{bmatrix}$$

Equation 25-66

where

$\tau_{S,a,,j}$ = auto space-based travel time rate (s/mi),

$\tau_{S,SUT,kin,j}$ = kinematic space-based travel time rate of SUTs (s/mi),

$\tau_{S,TT,kin,j}$ = kinematic space-based travel time rate of TTs (s/mi),

$\Delta\tau_{TI}$ = traffic interaction term (s/mi),

v_{mix} = flow rate of mixed traffic (veh/h/ln),

FFS = base free-flow speed of the basic freeway segment (mi/h),

P_{SUT} = proportion of SUTs in the traffic stream (decimal), and

P_{TT} = proportion of TTs in the traffic stream (decimal).

The traffic interaction term is the same for all the travel time rate equations and can be computed with Equation 25-62.

STEP 6: COMPUTE MIXED-FLOW SPACE-BASED TRAVEL TIME RATE AND SPEED

The mixed-flow space-based travel time rate $\tau_{\text{mix},j}$ and the space-based speed $S_{\text{mix},j}$ are computed with Equation 25-67 and Equation 25-68, respectively.

$$\tau_{\text{mix},j} = P_a \tau_{S,a,j} + P_{SUT}\tau_{S,SUT,j} + P_{TT}\tau_{S,TT,j}$$

Equation 25-67

$$S_{\text{mix},j} = \frac{3{,}600}{\tau_{\text{mix},j}}$$

Equation 25-68

where

$\tau_{\text{mix},j}$ = mixed-flow space-based travel time rate for segment j (s/mi),

$\tau_{S,a,j}$ = automobile space-based travel time rate for segment j (s/mi),

$\tau_{S,SUT,j}$ = space-based travel time rate of SUTs (s/mi),

$\tau_{S,TT,j}$ = space-based travel time rate of TTs (s/mi),

P_{SUT} = proportion of SUTs in the traffic stream (decimal), and

P_{TT} = proportion of TTs in the traffic stream (decimal).

As indicated above, Steps 3 through 6 are repeated for each segment until the end of the composite grade is reached.

STEP 7: OVERALL RESULTS

Once spot and space mean speeds and travel time rates have been developed for all vehicle types on all segments, the overall performance of the composite grade can now be estimated. The mixed-flow travel time for each segment can be computed with Equation 25-69.

Equation 25-69

$$t_{\text{mix},j} = \frac{3{,}600 d_j}{S_{\text{mix},j}}$$

where

$t_{\text{mix},j}$ = mixed-flow travel time segment j (s),

d_j = grade length of segment j (mi), and

$S_{\text{mix},j}$ = mixed-flow speed for segment j (mi/h).

The overall mixed-flow travel time $t_{\text{mix},oa}$ is the summation of mixed-flow travel times on all segments. The overall space-based travel speed can then be computed with Equation 25-70.

Equation 25-70

$$S_{\text{mix},oa} = \frac{3600 d_{oa}}{t_{\text{mix},oa}}$$

where

$S_{\text{mix},oa}$ = overall mixed-flow speed (mi/h);

d_{oa} = overall distance, the summation of all the segment grade lengths on the composite grade (mi); and

$t_{\text{mix},oa}$ = overall mixed-flow travel time (s).

8. FREEWAY CALIBRATION METHODOLOGY

This section presents a calibration methodology for the procedures described in Chapter 10, Freeway Facilities Core Methodology, and Chapter 11, Freeway Reliability Analysis. The freeway calibration methodology is carried out at three main levels:

1. Calibration at the core freeway facility level,
2. Calibration at the reliability level, and
3. Calibration at the Active Traffic and Demand Management (ATDM) strategy assessment level.

The procedure uses *sequential calibration* to calibrate these three distinct methodological parts, meaning that the calibration is carried out sequentially for each level. After a level is fully calibrated, no further change is allowed from a different level. As a result, this approach requires that the calibration parameters of different levels be mutually exclusive.

The approach first calibrates the base scenario, then focuses on reliability-level calibration, and concludes with ATDM-level calibration. It is logical both that the base scenario (i.e., core freeway facility) should be fully calibrated before evaluating reliability or ATDM strategies and that the base scenario calibration should not be affected by any subsequent changes from the reliability or ATDM calibration levels. Consequently, it is critical to select a suitable base scenario with oversaturated flow conditions to ensure that the bottlenecks are calibrated appropriately. More information about the development of the methodology is provided in a paper (*15*) located in the Technical Reference Library section of online HCM Volume 4.

Calibration relies on field measurements of key input variables, including the segment capacity. Chapter 12, Basic Freeway and Multilane Highway Segments, provides definitions for prebreakdown and queue discharge capacity. Chapter 26, Freeway and Highway Segments: Supplemental, provides guidance for field measuring and estimating capacity from sensor data.

CALIBRATION AT THE CORE FREEWAY FACILITY LEVEL

The core freeway facility analysis is calibrated for a specific day, called the *seed day.* Exhibit 25-26 depicts five steps of the calibration process for a core facility analysis. After gathering input data, the actual calibration consists of three steps (Steps 2, 3, and 4), the order of which is somewhat flexible. Multiple iterations may be needed to achieve satisfactory performance. A detailed explanation of each step follows.

Step 1: Gather Input Data

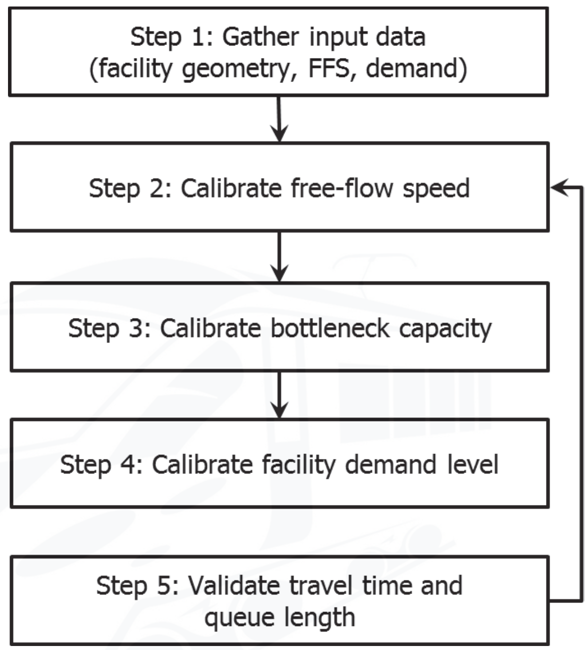
In this step, all input data required for a single freeway facility analysis (computational engine seed file) need to be gathered. These data include

1. Geometric information such as segment type, segment length, and number of lanes;

2. Facility free-flow speed (FFS);
3. Capacity estimate for bottleneck segment(s); and
4. Demand-level data for all segments in all time intervals.

Geometric data are model input parameters and will not be changed in the calibration process. The other three inputs (FFS, capacity, and demand) are used as calibration parameters.

Exhibit 25-26
Calibration Steps for the Core Freeway Facility Level



Step 2: Calibrate Free-Flow Speed

FFS can be field measured or estimated by using the procedure given in Chapter 12. The FFS calibration procedure may be applied in either case; however, if accurate field measurements of FFS are available, great care should be taken before changing a field-measured input.

To start, the analyst should select a time interval with a low demand level and no active bottleneck. The analyst should then compare the estimated free-flow travel time of this interval with the field measurements. Because a later step requires the analyst to look at congested periods, the study period should be sufficiently long to include free-flow conditions before or after the onset of congestion.

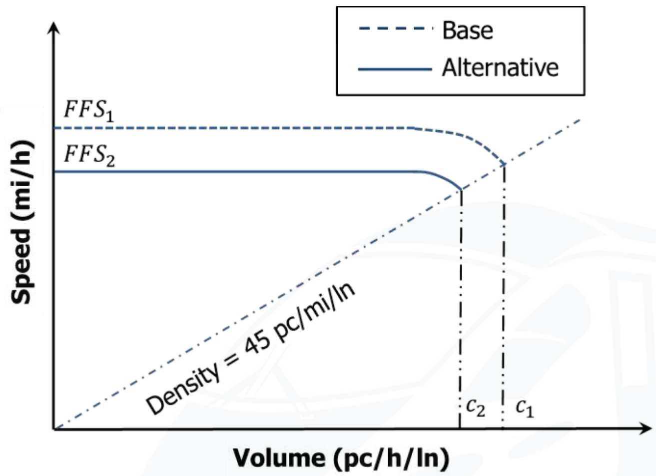
The calibration process involves making a computational engine run for the seed day, recording the average travel time for a low-demand time interval, and comparing it to the observed travel time. The user needs to repeatedly perform one of the following actions until the predicted facility travel time is within a predefined threshold (e.g., 10% error tolerance) of the observed facility travel time:

- *Reduce* the FFS in 1- to 5-mi/h increments if the predicted travel time is *less* than the observed travel time, or

- *Increase* the FFS in 1- to 5-mi/h increments if the predicted travel time is *more* than the observed travel time.

This process should only be used for analysis periods with demand levels far less than oversaturation (i.e., free-flow conditions). The speed–flow diagram in Exhibit 25-27 illustrates the effect of different FFSs on the overall facility speed–flow–density relationship. A higher free-flow speed FFS_1 and a lower free-flow speed FFS_2 are shown. A 5-mi/h drop in FFS is associated with a drop in capacity equal to 50 pc/h/ln, except at very high FFSs.

Exhibit 25-27
Effect of Calibrating Free-Flow Speed on Capacity



Step 3: Calibrate Bottleneck Capacity

In this step, the location and extent of bottlenecks are calibrated, which requires a freeway facility to feature at least some periods of oversaturated flow conditions. Guidance for selecting capacity measurement locations and for reducing the collected data is provided in Chapter 26.

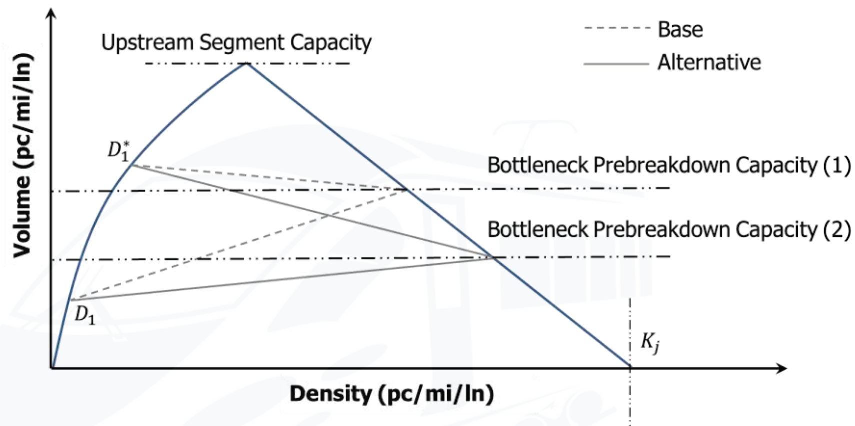
It is very important to calibrate for capacity, as research (*11*) shows the controlling capacity at the bottleneck is often significantly less than the HCM's base capacity. Three parameters are used to calibrate for the location and extent of bottlenecks:

1. *Prebreakdown capacity* at the bottleneck, implemented through a capacity adjustment factor (CAF) relative to the base capacity for a freeway segment. In the HCM, the prebreakdown flow rate is defined as the 15-min average flow rate immediately prior to the breakdown event. For the purposes of this chapter, the prebreakdown flow rate is equivalent to the segment capacity;
2. *Queue discharge rate* at the bottleneck following breakdown, as implemented through a percentage capacity drop α. In the HCM, the queue discharge rate is defined as the average flow rate during oversaturated conditions (i.e., during the time interval after breakdown and prior to recovery); and
3. *Jam density* of the queue forming upstream of the bottleneck, which describes the maximum density (minimum intervehicle spacing) in a queued condition.

The prebreakdown capacity and the queue-discharge capacity loss influence the actual throughput of the bottleneck, as well as the speed of shock waves describing the rate of change of the back of the queue. Jam density does not affect throughput; it only influences the formation and dissipation of queues at a bottleneck. The following exhibits illustrate the effects of these three calibration parameters in a shock wave diagram format.

In Exhibit 25-28, the number 1 denotes the base condition (dashed gray line) and the number 2 denotes the alternative condition (solid gray line). Two demand levels D are shown. Demand rates that are greater than the bottleneck capacity are noted with an asterisk.

Exhibit 25-28
Effects of Segment Capacity



Reducing the prebreakdown capacity increases the speed of the forming shock wave, but the speed of the recovery wave is decreased. As a result, a reduction in the segment's prebreakdown capacity is expected to increase congestion throughout the segment. Note that it is assumed a reduction in the segment capacity has no impact on the queue discharge rate at the bottleneck in the example above. The effects of a drop in queue discharge rate are shown in Exhibit 25-29.

Exhibit 25-29
Effects of Queue Discharge Rate Drop

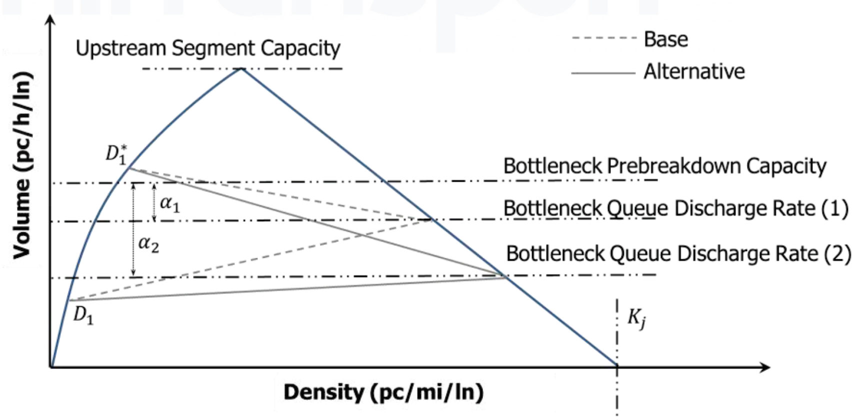
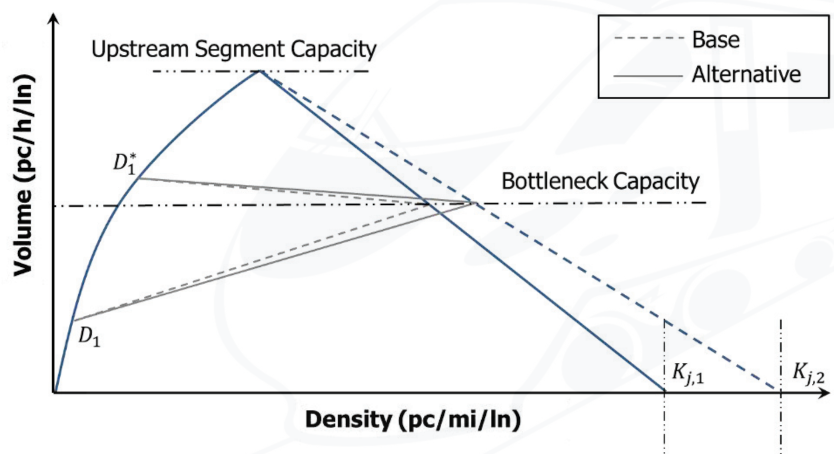


Exhibit 25-29 shows that including a queue discharge rate drop in the freeway model results in a reduction in bottleneck throughput after breakdown. The factor α describes the percentage reduction from prebreakdown capacity to queue discharge rate. A larger α corresponds to a larger drop and lower throughput. Implementing this factor results in a drop in throughput, an increase in the speed of the forming shockwave, and a decrease in the speed of the recovery wave. The result is a threefold effect that leads to a higher level of congestion, which has been demonstrated in the literature (*16*). It is therefore expected that the capacity drop has a nonlinear effect on the overall facility performance.

Exhibit 25-30 shows the effect of an increase in the jam density on wave speeds. Interestingly, an increase in the jam density value reduces both the forming and recovery wave speeds, thus canceling each other's effects to some degree. The opposite situation occurs if jam density is decreased, in which case both the forming and recovery speeds will increase. Although jam density is likely to affect the queue size (a higher jam density results in a smaller queue size), it may not influence travel time values as much as the prebreakdown capacity and queue discharge rate do.

Exhibit 25-30
Effects of Jam Density



To calibrate for bottlenecks, the analyst needs to change the capacity and capacity drop values for different segments of the freeway facility to recreate the bottlenecks that are observed in the field. Therefore, the analyst must first identify recurring bottlenecks in the field.

Next, the calibration process begins with setting the segment capacity to the HCM value for the facility's FFS (e.g., 2,400 pc/h/ln for a 70-mi/h FFS). A value of 7% for capacity drop is recommended.

If these initial values predict the bottleneck location correctly, the analysis proceeds to the validation step. If the model fails to identify a bottleneck, the analyst should reduce capacity in increments of 50 pc/h/ln until a bottleneck occurs. However, if the HCM model identifies a bottleneck that does not exist in the field, the analyst should increase capacity in increments of 50 pc/h/ln until the bottleneck disappears.

It is recommended that analysts wait to adjust the capacity drop value until after the bottleneck locations have been fixed. This procedure is performed as part of validating the queue length and travel time, as explained in Step 5.

Step 4: Calibrate Facility Demand Level

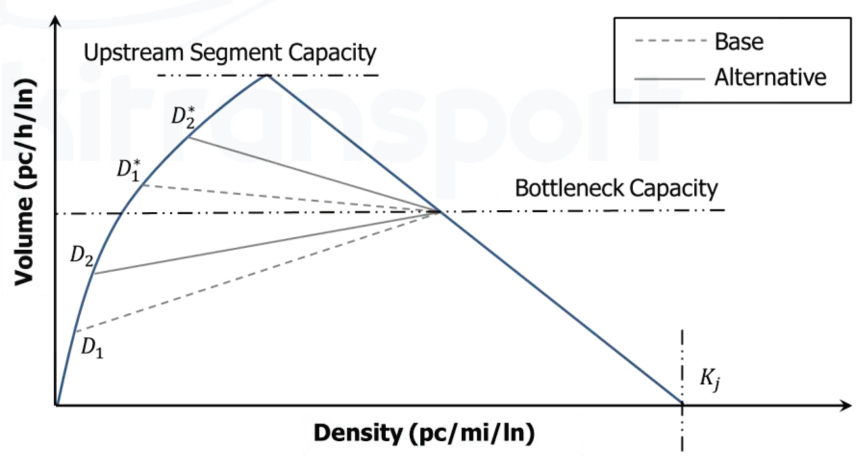
The demand level is a model input that can serve as a calibration parameter as a last resort. Presumably, demand has been measured based on field data, and therefore can be considered to be a fixed input. However, given the variability of demand (i.e., day-to-day fluctuation), as well as potential errors in volume and demand measurements, demand can become a calibration parameter after the FFS and capacity adjustment possibilities have been exhausted.

Two potential problems may be encountered with demand levels. First, in oversaturated conditions, it is not possible to measure the demand level downstream of a bottleneck or within a queued segment. The volume served is measured, rather than the demand level. Second, demand data vary from day to day, and the selected demand levels may not represent a “typical” day. This second problem is also true if AADT demand values are used to estimate peak period demands. As a result, although demand level is one of the inputs to the core freeway facility analysis, it may be subject to calibration.

To provide an example of the effect of the demand level on segment and facility travel time, a shockwave representation of the oversaturation model used in the core HCM freeway facilities methodology is presented. Although the HCM uses an adaptation of the cell-transmission model to estimate queue propagation and dissipation patterns at a bottleneck, the shockwave approach is useful to illustrate the calibration concepts here.

Exhibit 25-31 shows the flow–density relationship under high- and low-volume conditions for a segment that is just upstream of a bottleneck with a reduced capacity. As before, the number 1 denotes the base condition (dashed gray line), the number 2 denotes the alternative condition (solid gray line), and demand rates greater than the bottleneck capacity are denoted with an asterisk.

Exhibit 25-31
Effect of Demand Level



In Exhibit 25-31 it is evident that an overall increase in demand level (from D_1^* to D_2^* and from D_1 to D_2) would result in both an increase in the forming shock wave speed and a reduction in the recovery wave speed, assuming a fixed bottleneck capacity. In other words, an overall increase in demand level results in a higher level of congestion throughout. The greater the difference between upstream demand and downstream bottleneck capacity, the faster the resulting shock wave either grows the queue (demand-to-capacity ratio > 1.0) or dissipates the queue (demand-to-capacity ratio ≤ 1.0).

The analyst should increase the demand level in increments of 50 pc/h/ln until all bottlenecks that are observed in the field are activated in the freeway facility core analysis. However, if the model predicts bottlenecks that do not exist in the field, the user should decrease the demand level in increments of 50 pc/h/ln until those bottlenecks are deactivated. This activity should be performed in conjunction with Step 3: Calibrate Bottleneck Capacity.

Step 5: Validate Travel Time and Queue Length

The validation step has two major components:

1. Validate facility travel time, and
2. Validate queue length at active bottlenecks.

Travel Time Validation

After fixing the FFS and the bottleneck locations, the analyst should adjust the calibration parameters further to match predicted and observed facility travel times within a defined range (a 10% or less difference is recommended). Note that FFS has already been fixed in Step 3 and will not be adjusted further in this step. This process can be done by adjusting

1. Demand level,
2. Prebreakdown capacity,
3. Capacity drop, and
4. Jam density.

The analyst is trying to match reasonably well the estimated and observed facility and segment travel times. If the model *underestimates* the travel time, the analyst should consider one of the following actions:

1. Increase the demand level (in increments of 100 pc/h/ln),
2. Reduce prebreakdown capacity (in increments of 100 pc/h/ln), or
3. Increase the capacity drop (in increments of 1%).

If the model *overestimates* travel time, the analyst should consider one of the following actions:

1. Reduce the demand level (in increments of 50 pc/h/ln),
2. Increase prebreakdown capacity (in increments of 50 pc/h/ln), or
3. Reduce the capacity drop (in increments of 1%).

Note that jam density is unlikely to have a significant impact on facility travel time and is therefore not included in the steps above.

Queue Length Validation

After the facility travel time is fixed, the queue lengths at the facility's active bottlenecks should be matched reasonably well (i.e., within 10%) through further adjustments to the capacity drop and jam density.

If the predicted queue length at an active bottleneck is *shorter* than observed in the field, the capacity drop should be *increased* and the jam density should be *decreased*.

However, if the predicted queue length is *longer* than that observed in the field, the capacity drop should be *decreased* and the jam density should be *increased*. It is recommended that the capacity drop be changed in increments of 1% and that the jam density be changed in increments of 10 pc/mi/ln.

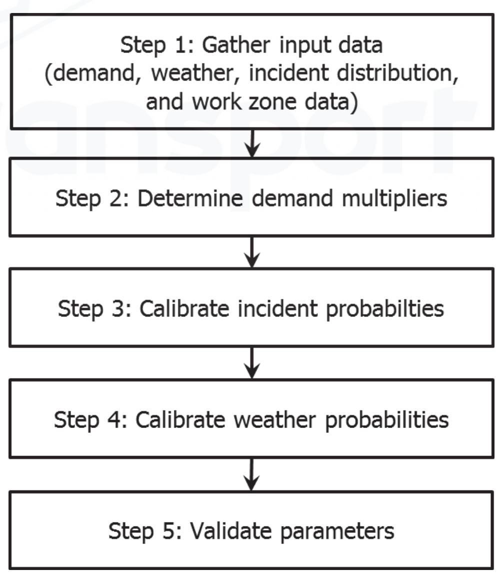
CALIBRATION AT THE TRAVEL TIME RELIABILITY LEVEL

After calibrating the core freeway facility methodology and fixing the value of its parameters, a comprehensive travel time reliability calibration is performed. Note that the process does not allow any change in the parameters that were calibrated in the previous step. The process requires a host of different input variables and calibration parameters. Comprehensive reliability-level calibration, as shown in Exhibit 25-32, starts with gathering the necessary input data. Some of these parameters, including facility geometry and FFS, are already known and fixed.

The process includes three major steps: whole-year demand calibration, incident calibration, and weather calibration. In the rest of this section, each step is presented in more detail.

To calibrate the methodology for a particular site, it is recommended that the analyst perform an initial comprehensive reliability run using default values for all input parameters and subsequently compare the predicted travel time index (TTI) cumulative distribution to the observed distribution. This section provides suggestions on how to change calibration parameters on the basis of the difference between the two TTI distributions.

Exhibit 25-32
Comprehensive Reliability Calibration Steps



Step 1: Gather Input Data

In this step, all the input data required for a reliability analysis are gathered. These data include

1. Demand distribution over the reliability reporting period, converted to monthly and day-of-week demand multipliers;
2. Incident or crash rates and event durations, with the corresponding speed and capacity adjustment factors;
3. Weather probabilities, with the corresponding speed and capacity adjustment factors; and
4. Work zone and special event data, with the corresponding speed and capacity adjustment factors.

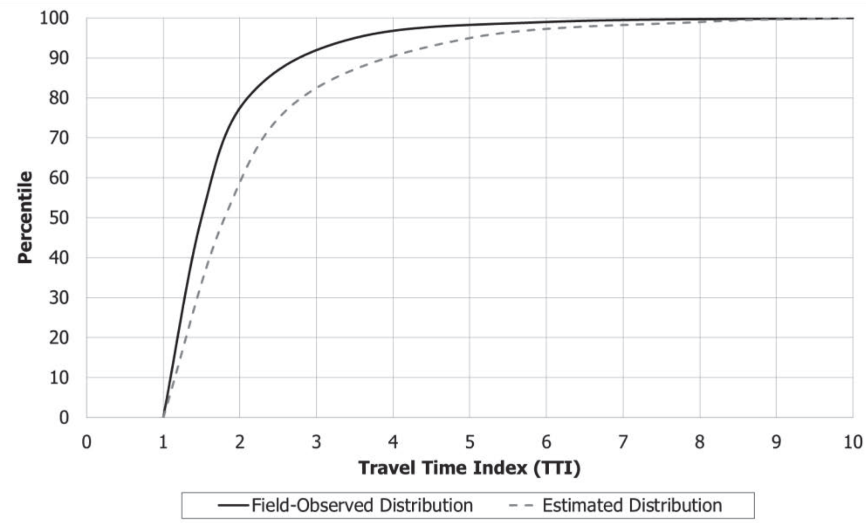
Specific details about these input data are provided in Chapter 11, Freeway Reliability Analysis.

Step 2: Determine Demand Multipliers

As mentioned above, the demand level for the seed day is either known or calibrated at the core freeway facility analysis level. However, in addition to the seed day, the reliability analysis requires the demand level for the other days included in the reliability reporting period. Because it is not feasible to measure demand level for all days, the methodology uses demand multipliers to convert the seed day demand to demand level for different days.

Although the demand level of the seed day may be accurately measured, the seed day may have experienced unusually low or high demand levels. In that event, the seed day demand either inflates or deflates the demand level for the other days of the reliability reporting period. In the example shown in Exhibit 25-33, a high demand level on the seed day causes the resulting TTI distribution to be consistently shifted to the right compared to the distribution observed in the field, across the full range of the distribution. Key reliability performance measures, such as TTI_{mean} or TTI_{95}, are also overestimated by the procedure in the case shown. To fix this problem (i.e., an inflated demand level for the seed day), the analyst needs to reduce the demand level in the seed file and make additional runs to determine whether the problem is resolved.

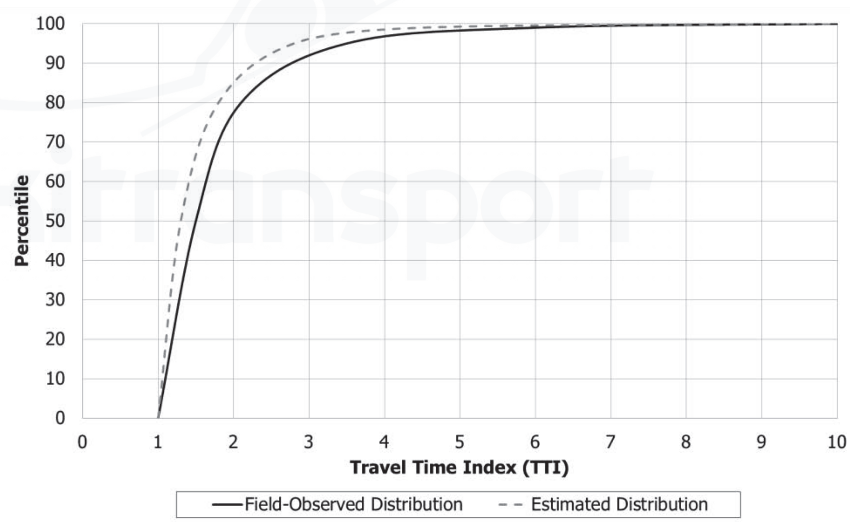
Exhibit 25-33
High Demand Level on the Seed Day



Note also in Exhibit 25-33 that the intercept with the x-axis is the same for both distributions, suggesting that the free-flow travel time at very low demands is the same in both cases. If the two distributions do not match at very low flow rates, this may be an indication that the free-flow speed calibration step for the core method was not performed correctly.

In contrast, in the example shown in Exhibit 25-34, the predicted TTI values are consistently lower than the observed values, suggesting that the seed day has an unusually low demand level. To resolve the problem, the demand level on the seed day should be increased and additional reliability runs performed.

Exhibit 25-34
Low Demand Level on the Seed Day



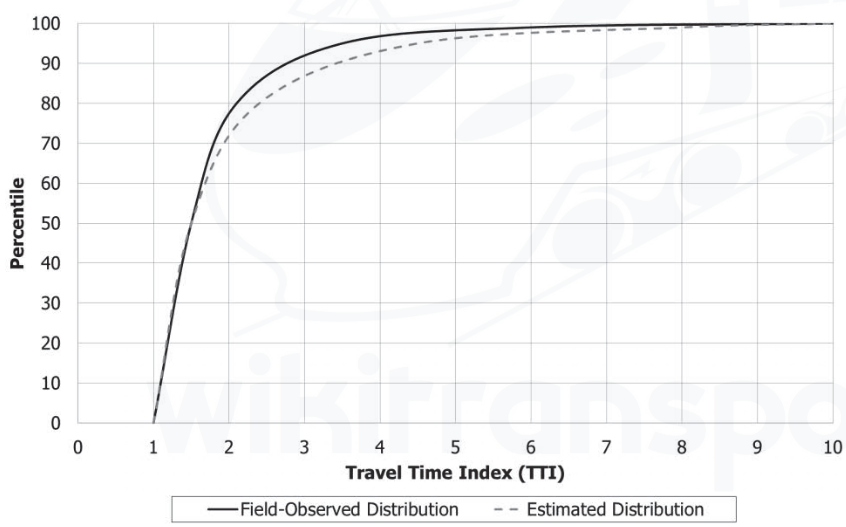
Another calibration lever is to change the distribution of the demand multipliers over the days of the reliability reporting period. This effort can improve the calibration of the methodology; however, its outcome is harder to predict. Users should change the distribution only when they have additional field information about seasonal and daily changes in the demand level that can bring it closer to reality.

When adjusting the demand level, users should try to bring the estimated 50th percentile TTI value to within 10% of the field-observed value. This is an iterative process that requires adjusting either the seed day demand level or the distribution of the demand multipliers, performing an additional comprehensive reliability run, and comparing the modeled and field-measured 50th percentile TTI values.

Step 3: Calibrate Incident Probabilities

When the demand level is calibrated, the predicted and observed TTI distributions are expected to closely follow each other up to the 50th to 60th TTI percentiles. However, nonrecurring sources of congestion usually influence the higher percentiles of the TTI distribution. They may cause a drift in distributions for higher percentiles, as shown in Exhibit 25-35. The figure shows a match between the predicted (red) and observed (blue) TTI distributions, but then suggests an overestimation of TTIs for higher percentiles with the red curve shifted to the right. As a result, to more accurately calibrate the comprehensive reliability analysis, the focus should be on incident and weather events. Incidents are known to have a more considerable impact on congestion level, and therefore the model is calibrated for incidents first, followed by weather events.

Exhibit 25-35
Overestimating the Impacts of Nonrecurring Sources of Congestion



Incidents can be calibrated by using a number of parameters as listed below:

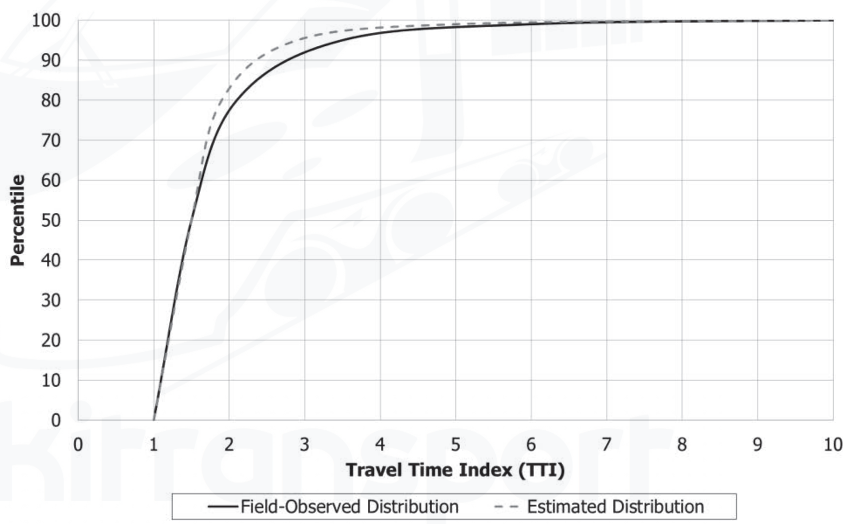
1. Probability of incident severity for each month, or crash rate per 100 million vehicle-miles traveled for each month and crash-to-incident rate and incident severity distribution, depending on the approach used for scenario generation;
2. Incident duration attributes by severity type (mean, standard deviation, and distribution);
3. Capacity and speed adjustment factors by severity type; and
4. Demand adjustment factors by severity type.

Incident attributes can be used to address overestimation in the tail of the predicted TTI distribution and to bring it closer to the observed distribution. For the example shown in Exhibit 25-35, the predicted and observed TTI distributions almost match each other up to the 60th TTI percentile, indicating that the demand level and base congestion level (i.e., recurring congestion) are calibrated well. After the 60th percentile, the reliability methodology overestimated TTI values in this case.

To reduce TTI values, the analyst should start by reducing the crash rate or incident probability. The same effect is expected by reducing the demand adjustment factor (for incidents). Note that in the case of severe incidents, a significant reduction in the demand level is expected, as drivers start to reroute to avoid the congestion. Finally, increasing the capacity and speed adjustment factors are expected to reduce the impacts of incidents as well.

On the other hand, if the method underestimates TTI values at the tail of the distribution (see Exhibit 25-36), the user can increase the crash rate, incident probability, or demand adjustment factor. (Note that the maximum allowable value for the demand adjustment factor is 1.) In addition, reducing capacity and speed adjustment factors for incidents is expected to magnify the impacts of incidents on travel time and consequently increase TTI values.

Exhibit 25-36
Underestimating the Impacts of Nonrecurring Sources of Congestion



Step 4: Calibrate Weather Probabilities

Similar to incidents, weather events influence the tail of the TTI distribution, but to a lesser extent. The following calibration parameters are available:

1. Probability of different weather events by month,
2. Duration of each weather event,
3. Capacity and speed adjustment factors, and
4. Demand adjustment factor.

These calibration parameters are expected to impact the TTI distribution similarly to those parameters mentioned in Step 3 for incident calibration. Note that weather information is more likely to be accurate as it is based on 10 years of data, while incident data are more difficult to gather. In addition, incidents have a more considerable impact on the TTI distribution. Therefore, as mentioned previously, it is recommended that the methodology be calibrated first through the demand and incident data, with the analyst turning to the weather-related parameters only if additional calibration is required.

For the example shown previously in Exhibit 25-35, the model overestimated TTI values in the tail of the distribution. The analyst can bring the two distributions closer to each other by reducing the probability of different weather events or by reducing their duration. The same effect is possible by increasing the capacity and speed adjustment factors or by reducing the demand adjustment factor. Note that in the case of extreme weather events, a significant reduction in the demand level is expected as travelers might decide to cancel their trips. However, data on such trends are very scarce and hard to collect. It is recommended that analysts adjust the demand adjustment factors only when there is evidence or knowledge of the trends on the study facility.

On the other hand, when the methodology underestimates TTI values in the tail of the distribution, as in Exhibit 25-36, the analyst can increase the probability of weather events or increase their durations. In addition, a reduction in capacity and speed adjustment factors is expected to move the distribution to the right.

Step 5: Validation

Changing all of the calibration parameters at the same time might lead to unexpected results. Therefore, the user is encouraged to change only one parameter at a time, run the comprehensive reliability methodology, plot and evaluate the new TTI distribution, and only then decide whether and how to change other parameters. The use of a computational engine makes running repeated reliability analyses with changing inputs a straightforward process.

The analyst should try to bring at least the predicted 80th and 95th percentile TTI values within 10% of the field-observed values. Preferably, additional percentiles should match the field data, although a perfect match may not be achievable. The collected field data should span the same reliability reporting period that was selected for the analysis, to ensure that results are comparable.

CALIBRATION AT THE RELIABILITY STRATEGY ASSESSMENT LEVEL

Calibration at the reliability strategy assessment level is only possible for strategies that have already been implemented in the field. For other strategies, calibration is not possible, other than based on expert judgment or comparison to an alternative tools analysis. However, the user can run a set of sensitivity analyses for each strategy to identify the trends and make sure that they match expectations. For example, a ramp-metering strategy is expected to shift the TTI distribution to the left, toward lower TTI values. The lower the metering rate, the larger the expected shift. If such a trend is observed, and if its extent is in a reasonable range, one can conclude that methodology works reasonably.

Similar to the calibration procedure at the comprehensive reliability level, the analyst must first gather all input data on facility geometry, free-flow speed, and demand level. Note that an important assumption is that the demand, incident, and weather calibration parameters are already fixed in the comprehensive reliability calibration step. As a result, the analyst is left with the remaining calibration parameters that are specific to each scenario.

In general, different scenarios may change a facility's free-flow speed, capacity, demand, incident probability, and average incident duration. Therefore, "scenario-specific" calibration parameters are

1. Speed adjustment factor,
2. Capacity adjustment factor,
3. Metering rate,
4. Demand adjustment factor,
5. Incident probability, and
6. Average incident duration.

It is recommended that the analyst make a reliability strategy assessment run based on a combination of field measurements and default values, plot the predicted TTI distribution, and then compare the result to the field observation. Similar to the comprehensive reliability calibration procedure, the analyst can then make changes in the calibration parameters to bring the predicted distribution closer to the observed one.

Based on the modifications that each strategy makes in the freeway methodology, the user can adjust the corresponding calibration parameters. Similar to calibrating the comprehensive reliability methodology, increasing the speed adjustment factor is expected to reduce travel time across the facility, while reducing it has an opposite effect. Increasing the value of the capacity adjustment factor is expected to reduce the facility travel time. Increasing the metering rate will allow more vehicles to enter the mainline and is expected to increase the facility travel time and perhaps activate bottlenecks in merge areas. On the other hand, reducing the metering rate is likely to reduce travel time across the facility and eliminate bottlenecks at merge areas. Increasing the demand adjustment factor is expected to increase travel time throughout the facility and shift the TTI distribution toward larger TTI values, while reducing it has the opposite effect. Increasing the incident probability is expected to shift the tail of the TTI distribution toward higher TTI values, while reducing it shifts the tail toward lower values. Finally, changing the average incident duration is expected to influence the TTI distribution similarly to incident probability.

The analyst should avoid making several changes in calibration parameters at the same time, as this may result in changes in TTI distribution that are hard to explain and may make the calibration procedure more difficult. Instead, analysts should select one calibration parameter at a time, make changes, rerun the strategy assessment procedure, plot the TTI distribution, compare it to the field distribution, and make other changes as necessary.

The user needs to first identify the main source of difference between the predicted and field TTI distributions. If a difference between the two distributions is observed throughout all ranges of TTIs (similar to Exhibit 25-33 and Exhibit 25-34), changing parameters such as the speed adjustment factor, capacity adjustment factor, demand adjustment factor, and metering rate is expected to bring the two distributions closer. The analyst should aim for a maximum of 10% difference between the 50th percentile of the predicted and observed TTI distributions at this stage.

On the other hand, if the difference between TTI distributions is observed mostly in the tail of the distribution (similar to Exhibit 25-35 and Exhibit 25-36), changing the incident probability and duration is expected to move the predicted distribution to the right. The analyst should aim for a maximum 10% difference between the 80th and 95th percentiles of the predicted and observed TTI distributions at this stage as well.

9. FREEWAY SCENARIO GENERATION

INTRODUCTION

This section provides details of the freeway scenario generation process. An overview of this process is provided in Chapter 11, Freeway Reliability Analysis, and elsewhere (*17*).

Freeway scenario generation utilizes a hybrid process, which includes deterministic and stochastic methods for modeling traffic demand, weather events, work zones, and incidents. The freeway reliability methodology uses a deterministic, calendar-based approach to model traffic demand levels and scheduled, significant work zone events. It uses a stochastic (Monte Carlo) approach to assign the occurrence of incident and weather events to scenarios. The method enumerates the different operational conditions on a freeway facility on the basis of varying combinations of factors affecting the facility travel time. Each unique set of operational conditions constitutes a *scenario.* A single replication of a scenario represents a unique combination of a day of week and month of year. The following seven principal stages, depicted in Exhibit 25-37, are involved in the scenario generation process:

- Stage 1, based on the user inputs, computes the number of different demand combinations and the resulting number of scenarios, along with their probabilities. These values also depend on the duration of the reliability reporting period.
- Stage 2 uses local traffic demand data to characterize the demand levels in the generated scenarios in a deterministic, calendar-based manner.
- Stage 3 incorporates scheduled work zones deterministically based on the calendar.
- Stage 4 incorporates published local weather event information, and generates the number and type of weather events, consistent with local data.
- Stage 5 randomly assigns the generated weather events in Stage 4 to the scenarios generated in Stage 1.
- Stage 6 utilizes the local crash or incident database to generate the number and severity of incident events, consistent with local data.
- Stage 7 randomly assigns incidents and their characteristics to each generated scenario in Stage 1.

The time frame within a given day when the reliability analysis is performed is called a *study period*. It consists of several contiguous 15-min *analysis periods*, which is the smallest temporal unit of analysis. The smallest spatial unit on the facility is an HCM analysis segment (see Chapters 12–14). The reliability reporting period is the time period over which the travel time distribution is generated (typically, but not necessarily, one year).

Each scenario representing a study period is characterized by a unique set of segment capacities, demands, free flow speeds, and number of lanes, for both general purpose and managed lane segments on the freeway facility. Various scenarios are created by adjusting one or more of the above parameters. A probability value is associated with each scenario that represents its likelihood of occurrence. This probability is computed on the basis of the number of scenarios and replications.

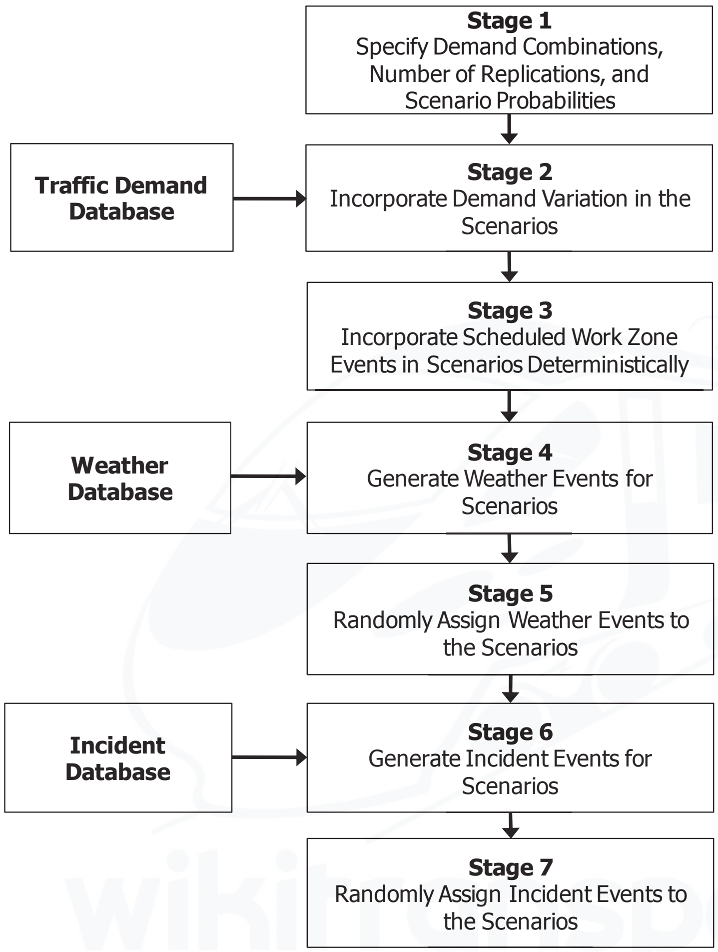
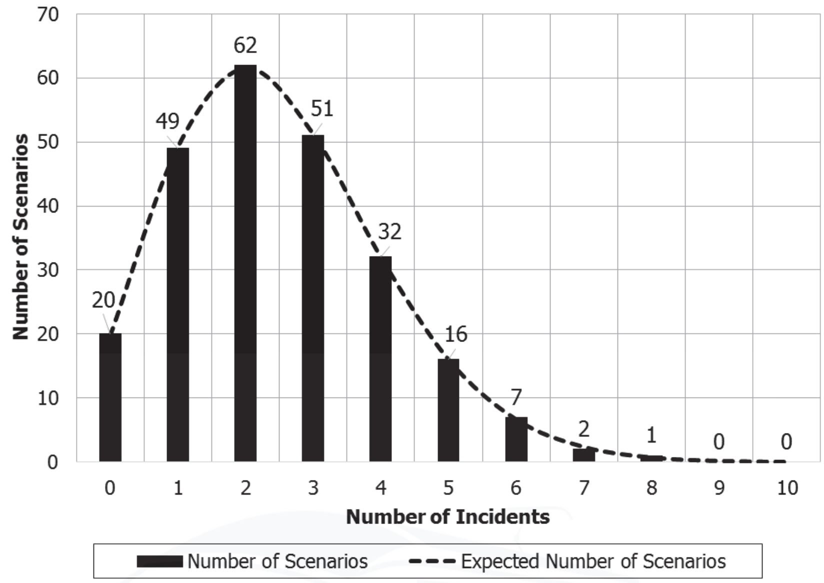


Exhibit 25-37
Process Flow Overview for Freeway Scenario Generation

Scenarios are generated in such a manner that the characteristics of the factors affecing travel time within scenarios best match the input, field-observed conditions. For example, the distribution of the number of incidents generated in various scenarios should yield a distribution similar to that observed in the field. Exhibit 25-38 depicts such an example, in which the number of incidents modeled in all scenarios (histogram) is designed to match field-observed values (curve).

Exhibit 25-38
Distribution of Number of Incidents in the Scenarios



Therefore, the process of generating scenarios effectively turns into an optimization problem. The objective is to maximize the match (or minimize the difference) between the predicted and field-observed distributions by assigning appropriate traffic demand levels, weather events, work zones, and incidents within the different scenarios. Eight distributions are considered in the scenario generation procedure:

1. Temporal distribution of traffic demand level (typically expressed as a ratio of scenario demand to AADT),
2. Temporal distribution of weather event frequency (by calendar month, randomly assigned to scenarios),
3. Distribution of average weather event duration by weather event type (by calendar month),
4. Temporal distribution of incident event frequency (by calendar month, weighted in the facility by segment VMT),
5. Distribution of incident severity (user specified),
6. Distribution of incident duration by severity (user specified),
7. Distribution of incident event start time (random), and
8. Spatial distribution of incident events (random).

The scenario generation method attempts to generate scenarios such that all eight specified distributions match field observations, with consideration for the need to round to integer values and to the 15-min duration of the analysis period. Such rounding is not likely to generate any significant systematic bias in the analysis.

METHODOLOGY

The freeway reliability scenario generation methodology consists of 34 steps. Exhibit 25-39 shows the methodology's process flow. Note that when managed lanes are present on the facility, the reliability scenarios should also consider their varying operational characteristics. The methodology assumes traffic demand levels and weather events affect both general purpose and managed lane operations simultaneously. However, the methodology does not account for scheduled work zone events on the managed lanes. Analysts should repeat Steps 19–34 should they desire to model incident events on the managed lanes separately. An explanation of each step in the process flow follows. All variables used in this section are defined in Section 2.

Step 1: Prepare Necessary Data for the Reliability Analysis

In this step, the analyst provides all necessary data for executing the scenario generation method. The starting point is preparing a complete seed file describing the facility's demand and geometry for a single study period. Developing the seed file is akin to developing a data set for the core methodology, as described in Chapter 10. In addition, for scenario generation purposes, additional data must include (*a*) the start and end clock times of the study period, (*b*) the duration of the reliability reporting period, (*c*) the seed file date, (*d*) the series of demand multipliers (see Step 4) for each demand combination, (*e*) the nearest metropolitan area to the facility (for weather station data), (*f*) the crash or incident rates by month of year on the facility, and (*g*) other local inputs.

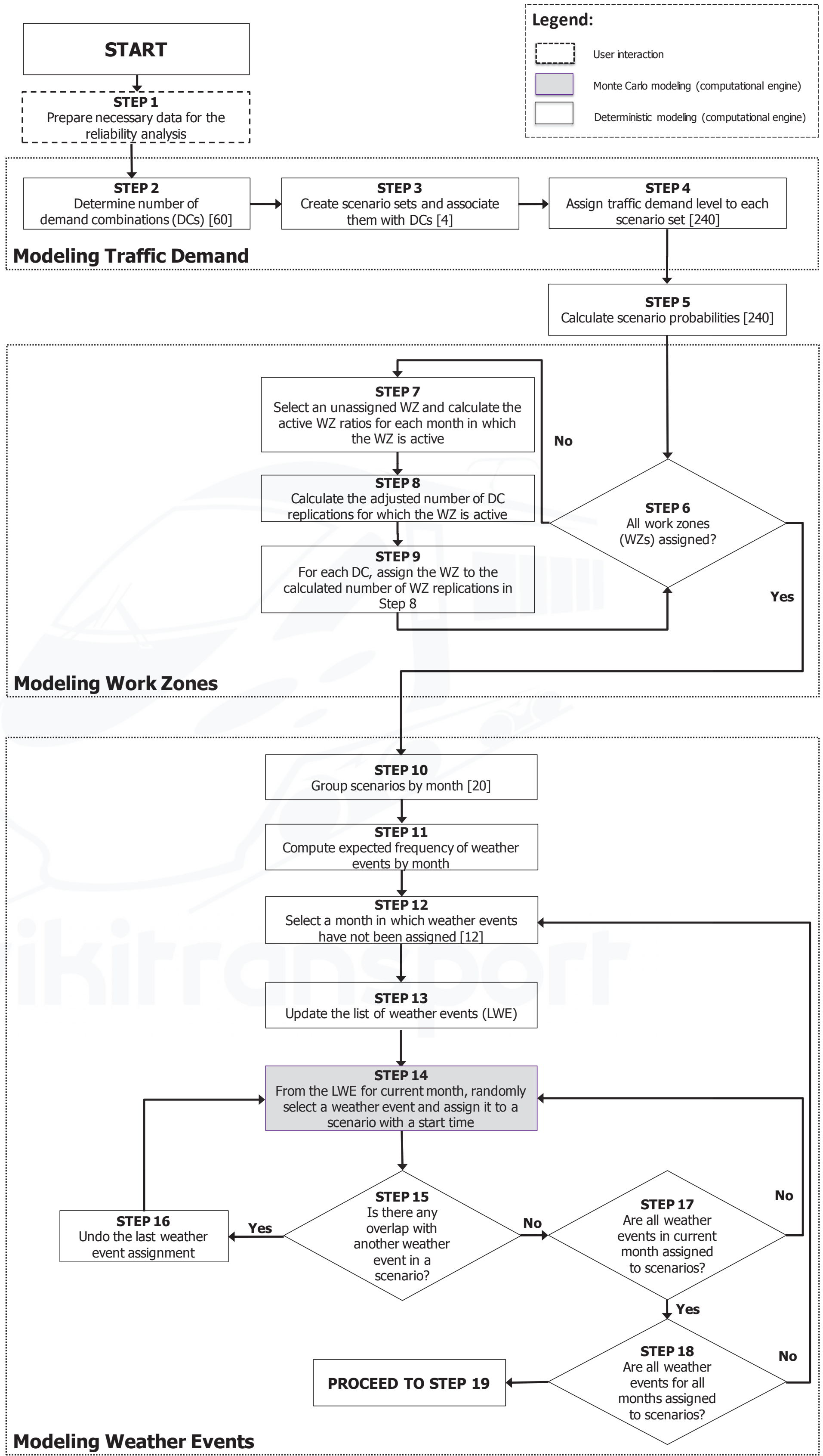
Step 2: Determine the Number of Demand Combinations

The freeway scenario generation method defines a demand combination as the combination of a specific weekday and month of year. Although demand levels in different demand combinations might be very similar (e.g., Tuesday and Wednesday afternoon volumes), the methodology handles them separately to keep the process simple. For a 1-year, weekday-only analysis, there are 60 such combinations (5 × 12). The number of demand combinations is defined by the variable N_{DC}.

Step 3: Create Scenario Sets and Associate Them with Demand Combinations

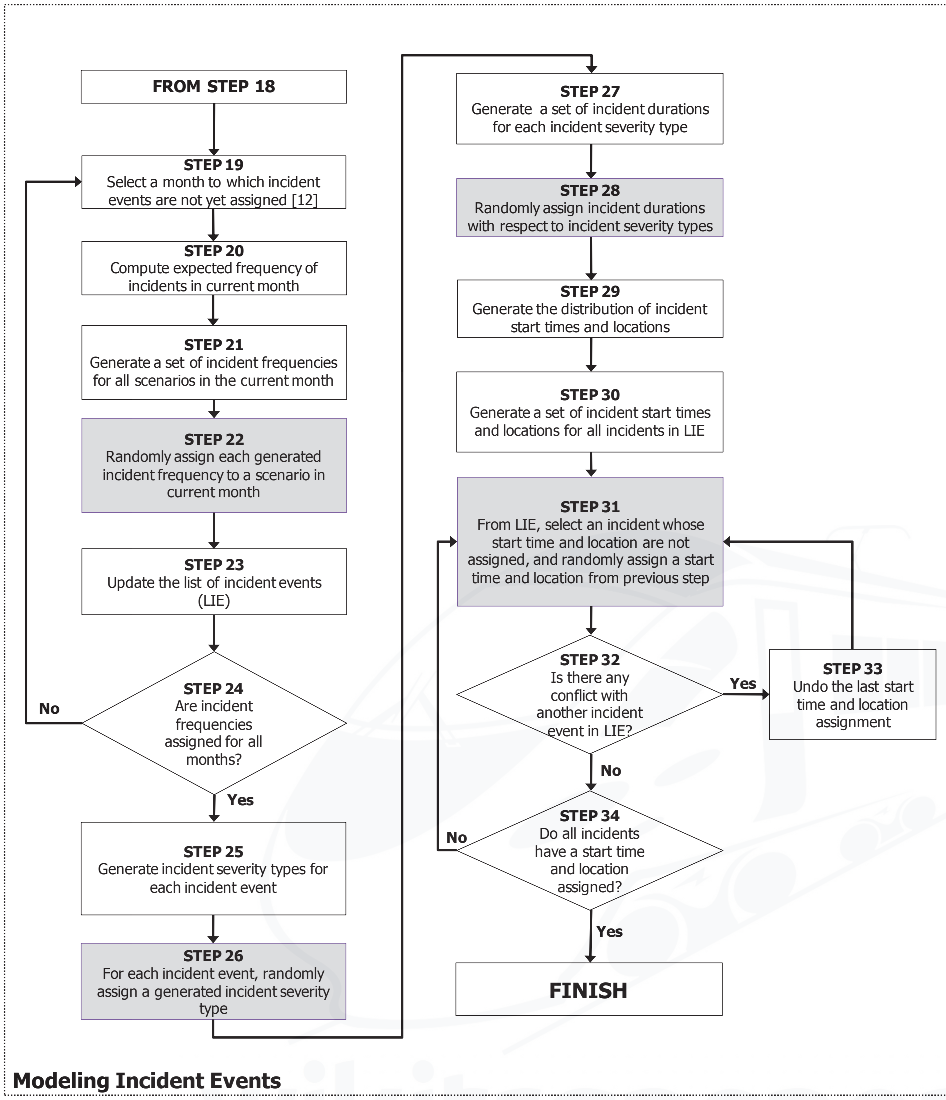
As a default, the methodology creates four scenario *replications* for each demand combination. The rationale behind four replications is that each demand combination usually consists of four or five calendar days. However, if a short-duration reliability reporting period is considered, the number of replications must be increased to capture sufficient variability in the travel time distribution. Typically, however, the default number of scenarios for a 1-year, weekday-only analysis would be 4 × 60 = 240 scenarios. The method allows the analyst to specify the number of replications per reliability analysis.

Exhibit 25-39
Detailed Freeway Scenario Generation Flowchart



Note: Numbers in brackets are default values.

Exhibit 25-39 (cont'd.)
Detailed Freeway Scenario Generation Flowchart



Note: Numbers in brackets are default values.

For each scenario, a set of adjustment factors is created for capacity, speed, demand, and number of lanes (CAF, SAF, DAF, and NLAF, respectively). At this point, each scenario contains default values for CAF, SAF, and DAF (all equal to 1) and NLAF (equal to 0), but the scenarios do not yet contain any demand, weather, or incident data. N_{Scen} represents the total number of scenarios and is computed as:

Equation 25-71

$$N_{Scen} = 4 \times N_{DC}$$

Step 4: Assign a Traffic Demand Level to Each Scenario Set

In this step, a traffic demand level is assigned to each scenario set (i.e., the number of replications used per scenario). For this purpose, demand multipliers, representing the ratio of the traffic demand level in each demand combination to the AADT are used to generate each scenario demand level. Because each scenario is associated with a unique demand combination, the ratio of the scenario demand multiplier to the seed file demand multiplier is used to determine the scenario demand, as shown in Equation 25-72.

Equation 25-72

$$DAF_s(tp, seg) = \frac{DM(s)}{DM(Seed_{tp})} \qquad \forall tp \in SP \text{ and } seg \in Segments$$

where

$DAF_s(tp, seg)$ = demand adjustment factor for scenario s, period tp, and segment seg;

$DM(Seed_{tp})$ = demand multiplier associated with the seed file; and

$DM(s)$ = demand multiplier associated with scenario s.

The process to calculate any demand value of any cell in a scenario is to multiply the cell demand value in the corresponding seed file (for the same HCM segment and analysis period) with the appropriate DAF, as shown in Equation 25-72. Note that if the facility contains managed lanes, the traffic demand level generated in this step will be effective for both the general purpose and managed lanes.

Step 5: Calculate Scenario Probabilities

The probability of a scenario occurrence is strictly a function of the number of days in the associated demand combination. Note that the probability of a scenario is fixed at this step and will not be altered in any subsequent steps. Simply stated, the probability of each scenario does not change by incorporating weather and incident events. The probability of each scenario is computed based on Equation 25-73.

Equation 25-73

$$P\{s\} = \frac{n_{\text{Day},DC_s}}{4 \times \sum_{k=1}^{N_{DC}} n_{\text{Day},k}}$$

where

$P\{s\}$ = probability of scenario s,

DC_s = demand combination associated with scenario s,

$n_{\text{Day},k}$ = number of days in the reliability reporting period associated with demand combination k (typically four for a 1-year weekday analysis), and

N_{DC} = number of demand combinations.

After computing each scenario's probability, the probabilities are assigned to the scenarios created in Step 3. The probability of a scenario is a function of the number of days in the associated demand combination, which is typically four or five for a whole-year analysis. For a typical 1-year, weekday-only analysis, the probability of each scenario is approximately 1/240 or 4.33%.

Step 6: Determine Whether All Work Zones Have Been Assigned

If there are no scheduled work zones during the reliability reporting period, or if all scheduled work zones have been assigned to scenarios, the process flow proceeds to Step 10. Otherwise, the process moves to Step 7 and assigns the next work zone. If there are no work zones considered in the reliability analysis, the process flow proceeds to Step 10.

Step 7: Calculate Active Work Zone Ratios

In this step, the parameter r_{DC} is calculated. This parameter is the ratio of each weekday type in which the work zone is active in a given month to the total number of each weekday type occurring in a given month. An unassigned work zone event is selected, and r_{DC} is calculated for each month in which the work zone is active.

Step 8: Calculate the Adjusted Number of Replications

For each affected demand combination in which a work zone is present, Equation 25-74 is used to calculate $\bar{N}_{DC,WZ}$, the adjusted number of replications of a demand combination for which the work zone is active.

$$\bar{N}_{DC,WZ} = \text{round}(r_{DC} \times N_r, 0)$$

Equation 25-74

Step 9: Assign the Work Zone to the Work Zone Replications

For each demand combination of each month in which the work zone is active, assign the work zone to the adjusted number of replications of each demand combination (equivalently scenarios) calculated in Step 8.

Step 10: Group Scenarios by Month

The attributes of inclement weather events are assumed to vary only by the month of the year. As such, in Step 10, all scenarios associated with a given month of year are grouped. Typically, this step involves grouping 20 scenarios (four replications of five weekdays each per month.)

Step 11: Compute the Expected Frequency of Weather Events by Month

The method uses the expected frequencies of weather events to create and characterize weather events. Historical data are used to estimate the probability, average duration, and standard deviation of duration of different weather conditions. Weather event likelihoods are reported in timewise probabilities that were computed for 103 metropolitan areas in the United States on the basis of 10 years of data. The resulting probability tables are provided as resource material in the Technical Reference Library in online HCM Volume 4. A listing of the 97 locations used to create the weather data is provided in Exhibit 25-40.

Only weather events that reduce capacity by more than 5% are included in the probability calculations. The average event duration and the standard deviation for each weather category are calculated by using the 10-year weather data set for each weather station. The probability of weather event type *i* in month *j* is found from Equation 25-75.

$$P_W\{i, j\} = \frac{\text{Sum of all SP durations in minutes in month } j \text{ that weather type } i \text{ is present}}{\text{Sum of all SP durations in minutes in month } j}$$

Equation 25-75

where SP indicates study period, and $P_W\{i, j\}$ is the probability of encountering weather type i in month j.

Exhibit 25-40
Listing of Weather Stations with Available Weather Data

#	Airport Code	City, State	#	Airport Code	City, State
1	KBHM	Birmingham, AL	**50**	KGSO	Greensboro, NC
2	KLIT	Little Rock, AR	**51**	KRIC	Raleigh, NC
3	KPHX	Phoenix, AZ	**52**	KOMA	Omaha, NE
4	KTUS	Tucson, AZ	**53**	KABQ	Albuquerque, NM
5	KBFL	Bakersfield, CA	**54**	KLAS	Las Vegas, NV
6	KFAT	Fresno, CA	**55**	KALB	Albany, NY
7	KLAX	Los Angeles, CA	**56**	KBUF	Buffalo, NY
8	KMOD	Modesto, CA	**57**	KLGA	New York, NY
9	KCMA	Oxnard, CA	**58**	KPOU	Poughkeepsie, NY
10	KROC	Riverside, CA	**59**	KSAC	Rochester, NY
11	KSAN	Sacramento, CA	**60**	KSYR	Syracuse, NY
12	KSAT	San Diego, CA	**61**	KCAK	Akron, OH
13	KSJC	San Francisco, CA	**62**	KCVG	Cincinnati, OH
14	KSLC	San Jose, CA	**63**	KCLE	Cleveland, OH
15	KSDF	Stockton, CA	**64**	KCMH	Columbus, OH
16	KCOS	Colorado Springs, CO	**65**	KDAY	Dayton, OH
17	KDEN	Denver, CO	**66**	KTOL	Toledo, OH
18	KBDL	Hartford, CT	**67**	KYNG	Youngstown, OH
19	KDCA	Washington, DC	**68**	KOKC	Oklahoma City, OK
20	KFMY	Cape Coral, FL	**69**	KTUL	Tulsa, OK
21	KJAX	Jacksonville, FL	**70**	KPDX	Portland, OR
22	KTPA	Lakeland, FL	**71**	KABE	Allentown, PA
23	KMIA	Miami, FL	**72**	KMDT	Harrisburg, PA
24	KSRQ	North Port, FL	**73**	KLNS	Lancaster, PA
25	KMCO	Orlando, FL	**74**	KPHL	Philadelphia, PA
26	KMLB	Palm Bay, FL	**75**	KPIT	Pittsburgh, PA
27	KATL	Atlanta, GA	**76**	KAVP	Scranton, PA
28	KAGS	Augusta, GA	**77**	KPVD	Providence, RI
29	PHNL	Honolulu, HI	**78**	KCHS	Charleston, SC
30	KDSM	Des Moines, IA	**79**	KCAE	Columbia, SC
31	KBOI	Boise City, ID	**80**	KGSP	Greenville, SC
32	KORD	Chicago, IL	**81**	KCHA	Chattanooga, TN
33	KIND	Indianapolis, IN	**82**	KTYS	Knoxville, TN
34	KICT	Wichita, KS	**83**	KMEM	Memphis, TN
35	KSEA	Louisville, KY	**84**	KBNA	Nashville, TN
36	KBTR	Baton Rouge, LA	**85**	KAUS	Austin, TX
37	KMSY	New Orleans, LA	**86**	KDFW	Dallas, TX
38	KBOS	Boston, MA	**87**	KELP	El Paso, TX
39	KCEF	Springfield, MA	**88**	KIAH	Houston, TX
40	KORH	Worcester, MA	**89**	KMFE	McAllen, TX
41	KBWI	Baltimore, MD	**90**	KSCK	San Antonio, TX
42	KPWM	Portland, ME	**91**	KOGD	Ogden, UT
43	KDTW	Detroit, MI	**92**	KPVU	Provo, UT
44	KGRR	Grand Rapids, MI	**93**	KRIV	Richmond, VA
45	KMSP	Minneapolis, MN	**94**	KORF	Virginia Beach, VA
46	KMCI	Kansas City, MO	**95**	KSFO	Seattle, WA
47	KSTL	St. Louis, MO	**96**	KMSN	Madison, WI
48	KJAN	Jackson, MS	**97**	KMKE	Milwaukee, WI
49	KCLT	Charlotte, NC			

Source: Zegeer et al. (*18*).

Equation 25-76 is used to convert those reported probabilities into rounded expected monthly weather event frequencies.

Equation 25-76

$$E[n_w, j] = \text{round}\left(\frac{P_t\{w, j\} \times D_{SP} \times N_{Scen,j}}{E_{15min}[D_w]}\right)$$

where

$E[n_w, j]$ = expected frequency of weather event w in month j, rounded to the nearest integer;

$P_t\{w, j\}$ = timewise probability of weather type w in month j;

D_{SP} = duration of study period SP (h);

$N_{Scen,j}$ = number of scenarios associated with month j of the reliability reporting period; and

$E_{15min}[D_w]$ = expected duration of weather event w rounded to the nearest 15-min increment.

In this step, the $E[n_w, j]$ values for each weather type w are computed in each month j of the reliability reporting period. Note that the unit of the expected frequency is *events per total scenario hours in each month*. Also note that the minimum value for $E_{15min}[D_w]$ is 0.25 h.

For example, if the study period is 5 h, if the probability of light rain during that month and time period (typically associated with about 20 scenarios) is 0.10, and if the average light rain event lasts 1 h, then the expected number of light rain events in that month is $(0.1 \times 5 \times 20)/1$, which rounds to 10 light rain weather events in that month, or 10 h of light rain in the month.

Step 12: Select a Month with Unassigned Weather Events

The process of assigning weather events in a month is independent of other months in the reliability reporting period. The process is carried out on a monthly basis. For this purpose, one month from the reliability reporting period without an assigned weather event is selected in the next steps.

Step 13: Update the List of Weather Events

In this step, the list of weather events is updated. That is, the weather events associated with the current month will have their characteristics (durations, CAFs, and SAFs) assigned.

Step 14: Assign Weather Events and Start Times to Scenarios

In this step, a weather event that was updated in the list of weather events in Step 13 is selected and randomly assigned to a scenario in the current month. The assignment of weather events to scenarios is carried out consistent with the relative scenario probabilities. In addition, a start time is randomly assigned to the selected weather event from the list of weather events. Because actual data on the start time of weather events are lacking, those are assigned randomly based on a uniform distribution.

Step 15: Identify Overlaps Between Weather Events in a Single Scenario

This step ensures there will be no temporal overlap between two weather events within a single scenario. Possible overlaps between weather events are checked, and if they exist, then Step 16 is executed. Otherwise, the process moves to Step 17.

Step 16: Undo the Most Recent Weather Event Assignment

If there is an overlap between weather events, the most recent weather assignment is undone. The process then goes back to Step 14 to reassign a scenario and a start time for the weather event.

Step 17: Check for Unassigned Weather Events in the Current Month

This step checks that all weather events present in the list of weather events have been assigned. If one or more unassigned weather events exist for the current month, the process returns to Step 14 to select another unassigned weather event.

Step 18: Check for Unassigned Weather Events in All Months

Once all weather events have been assigned to scenarios across all months in the reliability reporting period, the methodology proceeds to the incident modeling stage. Otherwise, the process returns to Step 12 to select another month from the reliability reporting period to have its weather events modeled in the associated scenarios.

Step 19: Select a Month with Unassigned Incidents

The methodology allows the user to directly enter monthly incident occurrences on a given facility during the study period into the procedure, should these values be available. Optimally, the distribution of incident durations, the start times, and the distribution of incidents by severity (e.g., number of lanes closed) could also be entered directly from a local incident database.

However, in most cases (including predictive reliability applications), these data will not be available, and incident events will need to be estimated from incident or crash rates (which vary by month and traffic demand levels). The methodology accounts for the correlation between incident and crash-only rates. Because the method attempts to generate the number of incident events based on their distributions, a high number of incidents could be assigned to a scenario that is associated with a low traffic demand level. The average traffic demand level for each month is therefore computed and used to characterize the incident events within scenarios in each month. Incident events are assigned to different months of the reliability reporting period independently. Therefore, a month from the reliability reporting period without any assigned incidents is first selected in the next steps.

Step 20: Compute the Expected Incident Frequency

The expected frequency of all incidents on the facility per study period in a given month j is computed with Equation 25-77.

Equation 25-77

$$n_j = IR_j \times VMT_j$$

where

n_j = expected frequency of all incidents in the study period for month j, rounded to the nearest integer;

IR_j = incident rate per 100 million VMT in month j; and

VMT_j = average vehicle miles traveled for scenarios in month j, after adjusting the demand in the base scenario with the appropriate demand multipliers and multiplying by the facility length in miles.

If IR_j is not locally available, Equation 25-78 can be used to estimate it.

$$IR_j = CR_j \times ICR$$

Equation 25-78

where CR_j is the local facilitywide crash rate per 100 million VMT in month j and ICR is the local incident-to-crash ratio. In the absence of other data, a national default value for ICR is 4.9.

When the crash rate is not available locally, the Highway Economic Requirements System (HERS) model can be used to estimate it (*19*). Agencies may also use other predictive models such as the *Highway Safety Manual* (*20*). The crash or incident rate is estimated per 100 million VMT. The HERS model uses Equation 25-79 to estimate the crash rate.

$$CR = (154.0 - 1.203 \times ACR + 0.258 \times ACR^2 - 0.00000524 \times ACR^5) \times e^{0.0082 \times (12 - LW)}$$

Equation 25-79

where CR is the crash rate per 100 million VMT, ACR is the facility AADT divided by its two-way hourly capacity, and LW is the lane width in feet.

Step 21: Generate a Set of Incident Frequencies

The distribution of the number of incidents in a study period can be characterized by a Poisson distribution. Assume there are $N_{Scen,j}$ scenarios (typically 20) associated with the current month j. Then, on average, $n_j \times N_{Scen,j}$ incidents (rounded to the nearest integer) need be to generated and assigned to scenarios. Therefore, a set of $N_{Scen,j}$ numbers should be generated that best matches a Poisson distribution with a mean value of n_j, per Equation 25-80.

For this purpose, an adjustment parameter δ_1 is defined. By solving Equation 25-80, the frequency of incidents for a set of $N_{Scen,j}$ scenarios can be computed, following the Poisson distribution. The values of the adjustment parameter usually hover around 1 and are estimated from the equality.

$$\sum_{k=0}^{+\infty} \left(\text{round}\left[\delta_1 \times N_{Scen,j} \times \text{Prob}\{n_{inc} = k\}\right]\right) = N_{Scen,j}$$

Equation 25-80

where n_{inc} is the number of incidents and other variables are as defined previously. Subsequently, the number of scenarios that are assigned k incidents ($k = 0 \rightarrow \infty$) is determined by Equation 25-81.

$$\text{Number of Scenarios with } k \text{ incident events} = \text{round}\left[\delta_1 \times N_{Scen,j} \times \text{Prob}\{n_{inc} = k\}\right]$$

Equation 25-81

where all variables are as defined previously. By setting different k-values in the above equation, a set of monthly incident frequencies will be generated in this step.

Step 22: Assign Incidents to Scenarios

The incidents generated in Step 21 are randomly assigned to the scenarios associated with the current month. A random number is drawn with respect to scenario probabilities to determine the assigned scenario number.

Step 23: Update the List of Incident Events

The list of incident events is updated after the incident frequencies are generated. This list holds information for each incident event in the entire reliability analysis. The associated incident event information includes the assigned scenario number, calendar month, incident duration, incident impact factors (e.g., CAF, SAF), incident segment location, and incident start time.

Step 24: Check for Unassigned Incidents

This step ensures that incident event frequencies are generated and assigned to scenarios for all months in the reliability reporting period. Once incidents in all months have been processed in Steps 20–23, the scenario generation process continues to Step 25.

Step 25: Generate Incident Severities for Each Incident Event

A set of incident severities is generated for the entire set of incidents developed in Step 21. Note that this step is not carried out on a monthly basis. The distribution of incident severities must be known a priori for incorporation in the methodology. This distribution is defined by $\mathbb{G}(i)$, which is assumed to be homogeneous across the facility and different demand levels.

Agencies can estimate this distribution by analyzing their incident logs or they can use national default values. Equation 25-82 gives the definition of $\mathbb{G}(i)$ as a discrete distribution, where i denotes the incident severity type (e.g., $i = 1$ is a shoulder closure, and $i = 5$ is a four-lane closure).

Equation 25-82

$$\mathbb{G}(i) = \begin{cases} g_1 & i = 1 \\ g_2 & i = 2 \\ g_3 & i = 3 \\ g_4 & i = 4 \\ g_5 & i = 5 \end{cases}$$

Suppose a total of $N_{Scen,Inc}$ incidents was generated in Steps 19–24. To generate incident severities, an adjustment parameter δ_2 is defined. By solving Equation 25-83, incident severities for all incidents in the list of incident events will be estimated that will follow the prespecified $\mathbb{G}(i)$ distribution.

Equation 25-83

$$\sum_i \left(\text{round}\left[\delta_2 \times N_{Scen,Inc} \times \mathbb{G}(i)\right]\right) = N_{Scen,Inc}$$

where all variables are as previously defined. The adjustment parameter is determined with Equation 25-83, and the number of scenarios that are assigned incident severity type i is determined by Equation 25-84.

Equation 25-84

$$\text{Number of incidents with severity } i = \left(\text{round}\left[\delta_2 \times N_{Scen,Inc} \times \mathbb{G}(i)\right]\right)$$

where all variables are as previously defined.

The distribution of incident severity $\mathbb{G}(i)$ is shown in Equation 25-85. These values are based on national default values (*18*).

$$\mathbb{G}(i)=\begin{cases}0.754 & i=1\text{ (shoulder closed)}\\ 0.196 & i=2\text{ (one lane closed)}\\ 0.031 & i=3\text{ (two lanes closed)}\\ 0.019 & i=4\text{ (three lanes closed)}\\ 0 & i=5\text{ (four or more lanes closed)}\end{cases}$$

Equation 25-85

Step 26: Assign Incident Severity Type

The incident severities generated in Step 25 are randomly assigned to the incidents in the list of incident events.

Step 27: Generate Incident Durations by Incident Severity Type

The duration of each incident severity type is assumed to follow a lognormal distribution (*15*). Exhibit 25-41 shows default parameters for the incident duration distribution developed through research (*18*).

Exhibit 25-41
Incident Duration Distribution Parameters in Minutes

Statistics	No. of Lanes Closed: Shoulder	1	2	3 or more
Range	8.7–58	16–58.2	30.5–66.9	36–93.3
Average	34.0	34.6	53.6	69.6
Median	36.5	32.6	60.1	67.9
Standard deviation	15.1	13.8	13.9	21.9

Because $N_{Inc,i}$ incidents are associated with severity i, a set of $N_{Inc,i}$ numbers can be generated that best matches a lognormal distribution of incident durations. For this purpose, an adjustment parameter δ_3 is defined, as shown in Equation 25-86.

$$\sum_t \left(\text{round}\left[\delta_3 \times N_{Inc,i} \times \text{Prob}\{Inc_{Dur}=t, Inc_{Type}=i\}\right]\right)=N_{Inc,i}$$

Equation 25-86

where Inc_{Dur} is the incident duration in minutes, Inc_{Type} is the incident severity type (1–5, as listed in Equation 25-85), and other variables are as defined previously.

By solving Equation 25-86, the adjustment parameter is determined. The number of scenarios that are assigned an incident duration t are then determined by Equation 25-87.

$$\text{Number of scenarios assigned incident severity } i = \text{round}\left[\delta_3 \times N_{Inc,i} \times \text{Prob}\{Inc_{Dur}=t, Inc_{Type}=i\}\right]$$

Equation 25-87

where all variables are as defined previously.

By inserting different t-values in Equation 25-87, a set of incident durations for each incident severity type will be generated.

Step 28: Randomly Assign Incident Durations by Severity

The incident durations generated in Step 27 are randomly assigned to the incidents in the list of incident events on the basis of the incident severity.

Step 29: Generate the Distribution of Incident Start Times and Locations

In this step, the distribution of each incident start time and location is assigned based on Step 20, with the likelihood of having an incident on a segment in a given analysis period being correlated to the segment VMT. The distribution of incident start times will coincide with the distribution of facility VMT across all analysis periods. Further, the distribution of the location of an incident will be similarly tied to the distribution of VMT for each segment across the study period. Since $VMT_{seg,u}$ represents the VMT on segment *seg* during analysis period u in the seed file, the distribution of the incident locations will be determined by Equation 25-88.

Equation 25-88

$$\text{Prob}\{Location = segment\ x\} = \frac{\sum_u VMT_{x,u}}{\sum_{seg,u} VMT_{v,u}}$$

where *Location* is the segment in which the incident occurs.

In a similar manner, the distribution of the incident start time will be determined by Equation 25-89.

Equation 25-89

$$\text{Prob}\{StartTime\ = analysis\ period\ y\} = \frac{\sum_v VMT_{v,y}}{\sum_{v,u} VMT_{v,u}}$$

where *StartTime* is the analysis period in which the incident starts.

Step 30: Generate Incident Start Times and Locations for All Incidents

Assuming there are $N_{Scen,Inc}$ incidents in the list of incident events, two sets of $N_{Scen,Inc}$ numbers should be generated that best match the incident start time and location distributions. For this purpose, two adjustment variables, δ_4 and δ_5, are defined by Equation 25-90 and Equation 25-91, respectively.

Equation 25-90

$$\sum_x \left(\text{round}\left[\delta_4 \times N_{Scen,Inc} \times \text{Prob}\{Location = x\}\right]\right) = N_{Scen,Inc}$$

Equation 25-91

$$\sum_y \left(\text{round}\left[\delta_5 \times N_{Scen,Inc} \times \text{Prob}\{StartTime = y\}\right]\right) = N_{Scen,Inc}$$

By solving Equation 25-90 and Equation 25-91, the adjustment parameters are determined. The number of incidents that are assigned to any segment *seg* are then determined from Equation 25-92.

Equation 25-92

$$\text{Number of incidents assigned to segment } seg\ = \text{round}\left[\delta_4 \times N_{Scen,Inc} \times \text{Prob}\{Location = seg\}\right]$$

Finally, the number of incidents that are assigned a starting time (analysis period *tp*) is determined from Equation 25-93.

Equation 25-93

$$\text{Number of incidents assigned a starting time in analysis period } tp\ = \text{round}\left[\delta_5 \times N_{Scen,Inc} \times \text{Prob}\{StartTime = tp\}\right]$$

By inserting different *seg* and *tp* values in the above equations, a set of incident locations and start times will be generated.

Step 31: Assign Start Times and Locations to Incidents

In this step, an incident from the list of incident events is selected whose start time and location have not been assigned. A start time and location already generated in Step 30 are randomly assigned to the selected incident.

Step 32: Check for Overlap with Previously Assigned Incidents

This step checks if there is any overlap between other incident events for which the start time and location have been assigned in the list of incident events. If there is an overlap, the process proceeds to Step 33. Otherwise, it proceeds to Step 34.

Step 33: Undo the Previous Start Time and Location Assignment

This step undoes the previous start time and location assignment from Step 31 that led to the identification of a conflict in the list of incident events in Step 32.

Step 34: Check Whether All Incident Start Times and Locations Have Been Assigned

If there are incidents in the list of incident events that have not been assigned a start time and location, the process returns to Step 31 for further assignment. Otherwise, all the incidents in the list of incident events have been fully described and are ready to be modeled in the scenarios.

10. COMPUTATIONAL ENGINE OVERVIEW

The FREEVAL-2015E computational engine is written in the Java programming language. Java is a free, open source, object-oriented programming language that is highly portable and will run on almost all platforms. Unlike procedural languages, which largely consist of code broken up into subroutines, object-oriented languages require that the code be expressed in terms of *objects*. These objects have functions that either operate on the data associated with them or on other objects. In Java, groups of objects are called *classes*. Classes are then grouped into *packages*, which seek to provide organization based on some shared purpose or similarity.

The computational engine consists of nine packages, each of which contains a group of classes specific to a certain aspect of the HCM analysis. The main package contains the two most important classes for the methodology. First, the Seed class contains all input data for the freeway facility (e.g., freeway geometry, demand) and is the backbone of the engine. Once the analysis has been run, the Seed class will also contain all output performance measures. Further, any reliability or ATDM analysis performed will use Seed as the basis for its analysis.

The second class in the main package is the GPMLSegment class. This class is used to represent the segments of the freeway facility (general purpose or managed lane), and contains the code for both the undersaturated and oversaturated computational modules. Much of this code is an exact translation of the HCM methodology, with differences only occurring when it was necessary to either improve the performance of the code, or to match Java programming conventions. An example of a difference is that some variable values may not be explicitly stored but rather are calculated only as needed.

The other eight packages build on these two main classes. Four of the packages consist of "helper" functions that are used throughout the code. These helper classes provide functionality ranging from general input-output actions, such as opening and saving files, to more specific purposes, such as creating facility output summaries and specifying parameters for ramp-metering methodologies. The final four packages relate to reliability and ATDM analysis. These packages contain the reliability scenario generator, as well as many additional data structures to facilitate data input for both reliability and ATDM analysis.

The Java programming language provides the integrated ability to generate its own documentation. Developers simply provide descriptions of classes, functions, and variables throughout the code, and Java compiles them into a set of documentation referred to as a "Javadoc." This Javadoc follows the format of the official documentation of the language, thus allowing it to be easily understood and used by anyone familiar with the language. This documentation has been generated and is packaged with the computational engine. A user guide for the graphical user interface version of the engine is available to provide guidance on its use. These items can be found in the Technical Reference Library in online HCM Volume 4.

11. EXAMPLE PROBLEMS

This section presents eleven example problems illustrating the evaluation of freeway facilities using the core methodology, the reliability methodology, and the ATDM methodology. Exhibit 25-42 presents a list of these problems.

Exhibit 25-42
List of Example Problems

Example Problem	Description	Application
1	Evaluation of an undersaturated facility	Operational analysis
2	Evaluation of an oversaturated facility	Operational analysis
3	Capacity improvements to an oversaturated facility	Operational analysis
4	Evaluation of an undersaturated facility with a work zone	Operational analysis
5	Evaluation of an oversaturated facility with a managed lane	Operational analysis
6	Planning-level analysis of a freeway facility	Planning analysis
7	Reliability evaluation of an existing freeway facility	Reliability analysis
8	Reliability analysis with geometric improvements	Reliability analysis
9	Evaluation of incident management	ATDM analysis
10	Planning-level reliability analysis	Planning analysis
11	Estimating freeway composite grade operations with the mixed-flow model	Specialized truck analysis

EXAMPLE PROBLEM 1: EVALUATION OF AN UNDERSATURATED FACILITY

The Facility

The subject of this operational analysis is a 6-mi-long urban freeway facility that is composed of 11 individual analysis segments, as shown in Exhibit 25-43.

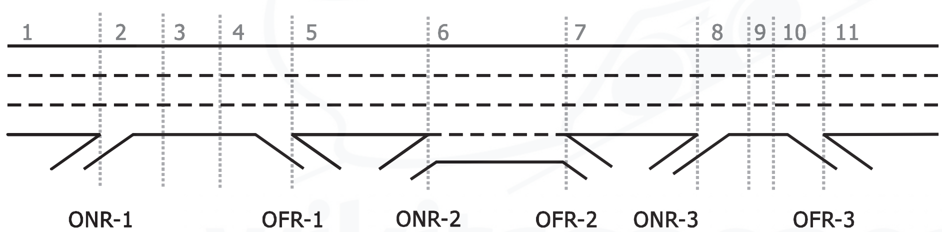


Exhibit 25-43
Example Problem 1: Freeway Facility

The facility has three on-ramps and three off-ramps. Geometric details are given in Exhibit 25-44.

Exhibit 25-44
Example Problem 1: Geometry of Directional Freeway Facility

Segment No.	1	2	3	4	5	6	7	8	9	10	11
Segment type	B	ONR	B	OFR	B	B or W	B	ONR	R	OFR	B
Segment length (ft)	5,280	1,500	2,280	1,500	5,280	2,640	5,280	1,140	360	1,140	5,280
No. of lanes	3	3	3	3	3	4	3	3	3	3	3

Notes: B = basic freeway segment; W = weaving segment; ONR = on-ramp (merge) segment; OFR = off-ramp (diverge) segment; R = overlapping ramp segment.

The on- and off-ramps in Segment 6 are connected by an auxiliary lane, and the segment may therefore operate as a weaving segment, depending on traffic patterns. The separation of the on-ramp in Segment 8 and the off-ramp in Segment 10 is less than 3,000 ft. Because the ramp influence area of on-ramps and off-ramps is 1,500 ft, according to Chapter 14, the segment affected by both ramps is analyzed as a separate overlapping ramp segment (Segment 9), labeled "R" in Exhibit 25-44.

The analysis question at hand is the following: What is the operational performance and LOS of the directional freeway facility shown in Exhibit 25-43?

The Facts

In addition to the information contained in Exhibit 25-43 and Exhibit 25-44, the following characteristics of the freeway facility are known:

SUTs and buses = 1.25% (all movements);

TTs = 1.00% (all movements);

Driver population = regular commuters;

FFS = 60 mi/h (all mainline segments);

Ramp *FFS* = 40 mi/h (all ramps);

Acceleration lane length = 500 ft (all ramps);

Deceleration lane length = 500 ft (all ramps);

D_{jam} = 190 pc/mi/ln;

c_{IFL} = 2,300 pc/h/ln (for *FFS* = 60 mi/h);

L_s = 1,640 ft (for Weaving Segment 6);

Total ramp density *TRD* = 1.0 ramp/mi;

Terrain = level; and

Analysis duration = 75 min (divided into five 15-min intervals).

A queue discharge capacity drop of 7% is assumed.

Comments

The facility was divided into analysis segments on the basis of the guidance given in Chapter 10, Freeway Facilities Core Methodology. The facility shown in Exhibit 25-43 depicts seven freeway *sections* (measured between ramps) that are divided into 11 analysis *segments*. The facility contains each of the possible segment types for illustrative purposes, including basic segment (B), weaving segment (W), merge segment (ONR), diverge segment (OFR), and overlapping ramp segment (R). The input data contain the required information needed for each of the segment methodologies.

The classification of the weave in Segment 6 is preliminary until it is determined whether the segment operates as a weave. For this purpose, the short length must be compared with the maximum length for weaving analysis to determine whether the Chapter 13, Freeway Weaving Segments, or the Chapter 12, Basic Freeway and Multilane Highway Segments, methodology is applicable. The short length of the weaving segment used for calculation is shorter than the weaving influence area over which the calculated speed and density measures are applied.

Chapter 12 must be consulted to find appropriate values for the heavy-vehicle adjustment factor f_{HV}. The computational engine automatically determines these adjustment factors for general terrain conditions, but user input is needed for specific upgrades and composite grades.

All input parameters have been specified, so default values are not needed. Fifteen-minute demand flow rates are given in vehicles per hour under prevailing conditions. These demands must be converted to passenger cars per hour under equivalent ideal conditions for use in the parts of the methodology related to segment LOS estimation. Details of the steps of the methodology follow.

Step A-1: Define Study Scope

In this initial step, the analyst defines the spatial extent of the facility (start and end points, total length) and the temporal extent of the analysis (number of 15-min analysis periods). The analyst should further decide which study extensions (if any) apply to the analysis (i.e., managed lanes, reliability, ATDM).

According to the inputs provided in the example, the number of time steps is five and the facility has 11 segments. The analysis does not involve a methodological extension.

Step A-2: Divide Facility into Sections and Segments

In this step, the analyst first defines the number of sections from gore point to gore point along the selected facility. These gore-to-gore sections are more consistent with modern freeway performance databases than HCM segments, and this consistency is critical for calibrating and validating the freeway facility. The analyst later divides sections into HCM segments (basic, merge, diverge, weave, overlapping ramp, or managed lane segment) as described in Chapter 10. The subject facility has already been segmented as shown in Exhibit 25-43.

Step A-3: Input Data

Data concerning demand, geometry, and other data are specified in this step. As the methodology builds on segment analysis, all data for each segment and each time period must be provided. Traffic demand inputs for all 11 segments and five analysis intervals are given in Exhibit 25-45.

Exhibit 25-45
Example Problem 1: Demand Inputs

Time Step (15 min)	Entering Flow Rate (veh/h)	Ramp Flow Rates by Time Period (veh/h)						Exiting Flow Rate (veh/h)
		ONR1	ONR2[a]	ONR3	OFR1	OFR2	OFR3	
1	4,505	450	540 (50)	450	270	360	270	5,045
2	4,955	540	720 (100)	540	360	360	270	5,765
3	5,225	630	810 (150)	630	270	360	450	6,215
4	4,685	360	360 (80)	450	270	360	270	4,955
5	3,785	180	270 (50)	270	270	180	180	3,875

Note: [a] Numbers in parentheses indicate ONR-2 to OFR-2 demand flow rates in Weaving Segment 6.

The volumes in Exhibit 25-45 represent the 15-min demand flow rates on the facility as determined from field observations or other sources. The actual volume served in each segment will be determined by the methodology. The demand flows are given for the extended time–space domain, consistent with the recommendations in Chapter 10. Peaking occurs in the third 15-min period. Because inputs are in the form of 15-min flow rates, no peak hour factor adjustment is necessary. Additional geometric and traffic-related inputs are as specified in Exhibit 25-44 and the Facts section of the problem statement.

Step A-4: Balance Demands

The traffic flows in Exhibit 25-45 are already given in the form of actual demands. Therefore, balancing demand is not necessary.

Step A-5: Identify Global Parameters

Global inputs are jam density and queue discharge capacity drop. Values for both parameters are given in the example problem's Facts section.

Step A-6: Code Base Facility

Step 6 is the first step requiring the use of a computational engine or software. Data input needs for the computational engine include all items collected or estimated in the previous steps. These data generally need to be entered for each segment and each time period, making this one of the most time-consuming steps in the analysis.

Step A-7: Compute Segment Capacities

Segment capacities are determined by using the methodologies of Chapter 12 for basic freeway segments, Chapter 13 for weaving segments, and Chapter 14 for merge and diverge segments. The resulting capacities are shown in Exhibit 25-46. Because the capacity of a weaving segment depends on traffic patterns, including the weaving ratio, it varies by time period. The remaining segment capacities are constant in all five time intervals. The capacities for Segments 1–5 and 7–11 are the same because the segments have the same basic cross section. The units shown are in vehicles per hour.

Exhibit 25-46
Example Problem 1: Segment Capacities

Time Step	Capacities (veh/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1						8,273					
2						8,281					
3	6,748	6,748	6,748	6,748	6,748	8,323	6,748	6,748	6,748	6,748	6,748
4						8,403					
5						8,463					

Step A-8: Calibrate with Adjustment Factors

This step allows the analyst to adjust demands, capacities, and FFSs for the purpose of calibration. The demand adjustment factor (DAF), capacity adjustment factor (CAF), and speed adjustment factor (SAF) can be modified for each segment and each time period. There is no adjustment needed for the subject facility according to the problem statement.

Step A-9: Adjust Managed Lane Cross Weave

This step is only required for facilities with managed lanes. The subject facility does not have a managed lane; therefore, this step is not required.

Step A-10: Compute Demand-to-Capacity Ratios

The demand-to-capacity ratios in Exhibit 25-47 are calculated from the demand flows in Exhibit 25-45 and the segment capacities in Exhibit 25-46.

Exhibit 25-47
Example Problem 1: Segment Demand-to-Capacity Ratios

Time Step	Demand-to-Capacity Ratios by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	0.67	0.73	0.73	0.73	0.69	0.63	0.72	0.79	0.79	0.79	0.75
2	0.73	0.81	0.81	0.81	0.76	0.71	0.81	0.89	0.89	0.89	0.85
3	0.77	0.87	0.87	0.87	0.83	0.77	0.89	0.99	0.99	0.99	0.92
4	0.69	0.75	0.75	0.75	0.71	0.61	0.71	0.77	0.77	0.77	0.73
5	0.56	0.59	0.59	0.59	0.55	0.47	0.56	0.60	0.60	0.60	0.57

The computed demand-to-capacity ratio matrix in Exhibit 25-47 shows no segments with a v_d/c ratio greater than 1.0 in any time interval. Consequently, the facility is categorized as *globally undersaturated*, and the analysis proceeds with computing the undersaturated service measures in Step A-11. Further, it is expected that no queuing will occur on the facility and that the volume served in each segment is identical to the input demand flows. Consequently, the matrix of volume-to-capacity ratios would be identical to the demand-to-capacity ratios in Exhibit 25-47. The resulting matrix of volumes served by segment and time interval is shown in Exhibit 25-48.

Exhibit 25-48
Example Problem 1: Volume-Served Matrix

Time Step	Volumes Served (veh/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	4,505	4,955	4,955	4,955	4,685	5,225	4,865	5,315	5,315	5,315	5,045
2	4,955	5,495	5,495	5,495	5,135	5,855	5,495	6,035	6,035	6,035	5,765
3	5,225	5,855	5,855	5,855	5,585	6,395	6,035	6,665	6,665	6,665	6,215
4	4,685	5,045	5,045	5,045	4,775	5,135	4,775	5,225	5,225	5,225	4,955
5	3,785	3,965	3,965	3,965	3,695	3,965	3,785	4,055	4,055	4,055	3,875

Step A-11: Compute Undersaturated Segment Service Measures

Because the facility is globally undersaturated, the methodology proceeds to calculate service measures for each segment and each time period, starting with the first segment in Time Step 1. The computational details for each segment type are exactly as described in Chapters 12, 13, and 14. The weaving methodology in Chapter 13 checks whether the weaving short length L_S is less than or equal to the maximum weaving length L_{max}. It is assumed, for any time interval where L_S is longer than L_{max}, that the weaving segment will operate as a basic freeway segment.

The basic performance measures computed for each segment and each time step are the segment speed (Exhibit 25-49), density (Exhibit 25-50), and LOS (Exhibit 25-51).

Exhibit 25-49
Example Problem 1:
Speed Matrix

Time Step	Speed (mi/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	60.0	53.9	59.7	56.1	60.0	48.0	59.9	53.4	53.4	56.0	59.7
2	59.9	53.2	58.6	55.8	59.6	46.8	58.6	52.3	52.3	55.7	57.6
3	59.4	52.6	57.2	55.7	58.3	46.2	56.2	50.6	50.6	51.8	55.1
4	60.0	53.8	59.7	56.1	60.0	49.7	60.0	53.6	53.6	56.0	59.9
5	60.0	54.9	59.8	56.3	60.0	52.5	60.0	54.8	54.8	56.5	60.0

Exhibit 25-50
Example Problem 1:
Density Matrix

Time Step	Density (veh/mi/ln) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	25.0	30.6	27.6	29.4	26.0	27.2	27.1	33.2	33.2	31.6	28.1
2	27.6	34.5	31.2	32.8	28.7	31.3	31.2	38.5	38.5	36.1	33.4
3	29.3	37.1	34.1	35.0	31.9	34.6	35.8	43.9	43.9	42.9	37.6
4	26.0	31.3	28.1	30.0	26.5	25.8	26.5	32.5	32.5	31.1	27.6
5	21.0	24.1	22.0	23.5	20.5	18.9	21.0	24.7	24.7	23.9	21.5

Exhibit 25-51
Example Problem 1:
LOS Matrix

Time Step	LOS by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	C	C	D	C	D	C	D	D	D	D	D
2	D	D	D	D	D	D	D	D	E	D	D
3	D	D	D	D	D	D	E	E	E	D	E
4	D	C	D	C	D	C	D	C	D	D	D
5	C	C	C	C	C	B	C	C	C	C	C

Step A-13: Apply Managed Lane Adjacent Friction Factor

This step is only required for facilities with managed lanes.

Step A-14: Compute Lane Group Performance

This step is only required for facilities with managed lanes.

Step A-15: Compute Freeway Facility Service Performance Measures by Time Interval

In this analysis step, facilitywide performance measures are calculated for each time step. Example calculations are provided for the first time step only; summary results are shown for all five time steps.

First, the facility space mean speed S is calculated for time step $t = 1$ from the 11 individual segment flows $SF(i, t)$, segment lengths $L(i)$, and space mean speeds in each segment and time step $U(i, t)$.

$$S(t = 1) = \frac{\sum_{i=1}^{11} SF(i,1) \times L(i)}{\sum_{i=1}^{11} SF(i,1) \times \frac{L(i)}{U(i,1)}}$$

$$\begin{aligned}\sum_{i=1}^{11} SF(i,1) \times L(i) &= (4{,}505 \times 5{,}280) + (4{,}955 \times 1{,}500) + (4{,}955 \times 2{,}280) + \\ &\quad (4{,}955 \times 1{,}500) + (4{,}685 \times 5{,}280) + (5{,}225 \times 2{,}640) + \\ &\quad (4{,}865 \times 5{,}280) + (5{,}315 \times 1{,}140) + (5{,}315 \times 360) + \\ &\quad (5{,}315 \times 1{,}140) + (5{,}045 \times 5{,}280) \\ &= 154{,}836{,}000 \text{ veh-ft}\end{aligned}$$

$$\sum_{i=1}^{11} SF(i,1) \times \frac{L(i)}{U(i,1)}$$

= (4,505 × 5,280 / 60.00) + (4,955 × 1,500 / 53.90)
+ (4,955 × 2,280 / 59.70) + (4,955 × 1,500 / 56.10)
+ (4,685 × 5,280 / 60.00) + (5,225 × 2,640 / 48.00)
+ (4,865 × 5,280/ 59.90) + (5,315 × 1,140/ 53.40)
+ (5,315 × 360 / 53.40) + (5,315 × 1,140 / 56.00)
+ (5,045 × 5,280 / 59.70)
= 2,688,024 veh-ft/mi/h

$$S(t=1) = \frac{154{,}836{,}000}{2{,}688{,}024} = 57.6 \text{ mi/h}$$

Second, the average facility density is calculated for Time Step 1 from the individual segment densities *D*, segment lengths *L*, and number of vehicles in each segment *N*.

$$D(t=1) = \frac{\sum_{i=1}^{11} D(i,1) \times L(i) \times N(i,1)}{\sum_{i=1}^{11} SL(i)N(i,1)}$$

$$\sum_{i=1}^{11} D(i,1) \times L(i) \times N(i,1)$$

= (25.6 × 5,280 × 3) + (31.3 × 1,500 × 3) + (28.2 × 2,280 × 3)
+ (30.1 × 1,500 × 3) + (26.6 × 5,280 × 3) + (27.8 × 2,640 × 4)
+ (27.7 × 5,280 × 3) + (33.9 × 1,140 × 3) + (33.9 × 360 × 3)
+ (32.4 × 1,140 × 3) + (28.8 × 5,280 × 3)
= 2,687,957 (veh/mi/ln)(ln-ft)

$$\sum_{i=1}^{11} SL(i)N(i,1)$$

= (5,280 × 3) + (1,500 × 3) + (2,280 × 3) + (1,500 × 3)
+ (5,280 × 3) + (2,640 × 4) + (5,280 × 3) + (1,140 × 3)
+ (360 × 3) + (1,140 × 3) + (5,280 × 3)
= 97,680 ln-ft

$$D(t=1) = \frac{2{,}747{,}253}{97{,}680} = 28.1 \text{ veh/mi/ln}$$

These calculations are repeated for all five time steps. The overall space mean speed across all time steps is calculated as follows:

$$S(p=5) = \frac{\sum_{p=1}^{5}\sum_{i=1}^{11} SF(i,p) \times L(i)}{\sum_{p=1}^{5}\sum_{i=1}^{11} SF(i,p) \times \frac{L(i)}{U(i,p)}}$$

The overall average density across all time steps is calculated as follows:

$$D(p=5) = \frac{\sum_{p=1}^{5}\sum_{i=1}^{11} D(i,p) \times L(i) \times N(i,p)}{\sum_{p=1}^{5}\sum_{i=1}^{11} SL(i)N(i,p)}$$

The resulting performance and service measures for Time Steps 1–5 and the facility totals are shown in Exhibit 25-52.

Exhibit 25-52
Example Problem 1: Facility Performance Measure Summary

Time Step	Performance Measure: Space Mean Speed (mi/h)	Performance Measure: Average Density (veh/mi/ln)	LOS
1	57.6	27.5	D
2	56.6	31.3	D
3	55.0	34.8	E
4	57.9	27.5	D
5	58.4	21.4	C
Total	**56.9**	**28.4**	**—**

Step A-16: Aggregate to Section Level and Validate Against Field Data

This step is used to validate the analysis and is performed only when field data are available.

Step A-17: Estimate LOS and Report Performance Measures for Lane Groups and Facility

The LOS for each time interval is determined directly from the average density for each time interval by using Exhibit 10-7. No LOS is defined for the average across all time intervals.

Discussion

This facility turned out to be globally undersaturated. Consequently, the facility-aggregated performance measures could be calculated directly from the individual segment performance measures. An assessment of the segment service measures across the time–space domain can begin to highlight areas of potential congestion. Visually, this process can be facilitated by plotting the v_d/c, v_a/c, speed, or density matrices in contour plots.

EXAMPLE PROBLEM 2: EVALUATION OF AN OVERSATURATED FACILITY

The Facility

The facility used in Example Problem 2 is identical to the one in Example Problem 1, which is shown in Exhibit 25-43 and Exhibit 25-44.

The Facts

In addition to the information in Exhibit 25-43 and Exhibit 25-44, the following characteristics of the freeway facility are known:

SUTs and buses = 1.25% (all movements);

TTs = 1.00% (all movements);

Driver population = regular commuters;

FFS = 60 mi/h (all mainline segments);

Ramp *FFS* = 40 mi/h (all ramps);

Acceleration lane length = 500 ft (all ramps);

Deceleration lane length = 500 ft (all ramps);

D_{jam} = 190 pc/mi/ln;

c_{IFL} = 2,300 pc/h/ln (for *FFS* = 60 mi/h);

L_s = 1,640 ft (for Weaving Segment 6);

TRD = 1.0 ramp/mi;

Terrain = level;

Analysis duration = 75 min (divided into five 15-min time steps); and

Demand adjustment = +11% increase in demand volumes across all segments and time steps relative to Example Problem 1.

As before, a queue discharge capacity drop of 7% is assumed.

Comments

The facility and all geometric inputs are identical to Example Problem 1. The same general comments apply. The results of Example Problem 1 suggested a globally undersaturated facility, but some segments were close to their capacity (v_d/c ratios approaching 1.0). In the second example, a facilitywide demand increase of 11% is applied to all segments and all time periods. Consequently, it is expected parts of the facility may become oversaturated and queues may form on the facility.

Step A-1: Define Study Scope

Similar to Example Problem 1, there are five time steps and the facility has 11 segments. The analysis does not include any extensions such as managed lanes, reliability, ATDM, or work zones.

Step A-2: Divide Facility into Sections and Segments

The subject facility segmentation is given in Exhibit 25-43. Therefore, there is no need to go through the segmentation process.

Step A-3: Input Data

The revised traffic demand inputs for all 11 segments and five analysis intervals are shown in Exhibit 25-53.

Exhibit 25-53 Example Problem 2: Demand Inputs

Time Step (15 min)	Entering Flow Rate (veh/h)	Ramp Flow Rates by Time Period (veh/h)						Exiting Flow Rate (veh/h)
		ONR1	ONR2[a]	ONR3	OFR1	OFR2	OFR3	
1	5,001	500	599 (56)	500	300	400	300	5,600
2	5,500	599	799 (111)	599	400	400	300	6,399
3	5,800	699	899 (167)	699	300	400	500	6,899
4	5,200	400	400 (89)	500	300	400	300	5,500
5	4,201	200	300 (56)	300	300	200	200	4,301

Note: [a] Numbers in parentheses indicate ONR-2 to OFR-2 demand flow rates in Weaving Segment 6.

The values in Exhibit 25-53 represent the adjusted demand flows on the facility as determined from field observations or demand projections. The actual volume served in each segment will be determined during the application of the methodology and is expected to be less downstream of a congested segment. The demand flows are given for the extended time–space domain, consistent with the methodology presented in Chapter 10. Peaking occurs in the third 15-min period. Because inputs are in the form of 15-min observations, no peak hour factor adjustment is necessary. Additional geometric and traffic-related inputs are as specified in Exhibit 25-44 and the Facts section of the problem statement.

Step A-4: Balance Demands

The traffic flows in Exhibit 25-53 have already been given in the form of actual demands and no balancing is necessary.

Step A-5: Identify Global Parameters

Global inputs are jam density and queue discharge capacity drop. Values for both parameters are given in the Facts section of the problem statement.

Step A-6: Code Base Facility

In this step, all input data for the subject are coded in the computational engine. Note that this facility can be coded by increasing entry demand across the facility by 11% relative to the Example Problem 1 demands.

Step A-7: Compute Segment Capacities

Because no changes to segment geometry were made, the segment capacities for basic and ramp segments are consistent with Example Problem 1. Capacities for weaving segments are a function of weaving flow patterns, and the increased demand flows resulted in slight changes as shown in Exhibit 25-54.

Exhibit 25-54
Example Problem 2:
Segment Capacities

Time Step	Capacities (veh/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1						8,273					
2						8,281					
3	6,748	6,748	6,748	6,748	6,748	8,323	6,748	6,748	6,748	6,748	6,748
4						8,403					
5						8,463					

Step A-8: Calibrate with Adjustment Factors

This step allows the analyst to adjust demands, capacities, and FFSs for the purpose of calibration. There is no adjustment needed for the subject capacity according to the problem statement.

Step A-9: Adjust Managed Lane Cross Weave

This step is only required for facilities with managed lanes. The subject facility does not have a managed lane.

Step A-10: Compute Demand-to-Capacity Ratios

The demand-to-capacity ratios in Exhibit 25-55 are calculated from the demand flows in Exhibit 25-53 and the segment capacities in Exhibit 25-54.

Exhibit 25-55
Example Problem 2: Segment Demand-to-Capacity Ratios

Time Step	Demand-to-Capacity Ratios by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	0.74	0.82	0.82	0.82	0.77	0.70	0.80	0.87	0.87	0.87	0.83
2	0.82	0.90	0.90	0.90	0.84	0.78	0.90	0.99	0.99	0.99	0.95
3	0.86	0.96	0.96	0.96	0.92	0.85	0.99	**1.10**	**1.10**	**1.10**	**1.02**
4	0.77	0.83	0.83	0.83	0.79	0.68	0.79	0.86	0.86	0.86	0.82
5	0.62	0.65	0.65	0.65	0.61	0.52	0.62	0.67	0.67	0.67	0.64

The computed v_d/c matrix in Exhibit 25-55 shows Segments 8–11 have v_d/c ratios greater than 1.0 (bold values). Consequently, the facility is categorized as *oversaturated*, and the analysis proceeds with computing the oversaturated service measures in Step A-12. It is expected that queuing will occur on the facility upstream of the congested segments and that the volume served in each segment downstream of the congested segments will be less than its demand. This residual demand will be served in later time intervals, provided the upstream demand drops and queues are allowed to clear.

Step A-12: Compute Oversaturated Segment Service Measures

Computations for oversaturation apply to any segment with a v_d/c ratio greater than 1.0 as well as any segments upstream of those segments that experience queuing as a result of the bottleneck. All remaining segments are analyzed by using the individual segment methodologies of Chapters 12, 13, and 14, as applicable, with the caveat that volumes served may differ from demand flows.

Similar to Example Problem 1, in Example Problem 2 the methodology calculates performance measures for each segment and each time period, starting with the first segment in Time Step 1. The computations are repeated for all segments for Time Steps 1 and 2 without encountering a segment with $v_d/c > 1.0$. Once the methodology enters Time Period 3 and Segment 8, the oversaturated computational module is invoked.

At the first active bottleneck, the v_a/c ratio for Segment 8 will be exactly 1.0 and the segment will process traffic at its capacity. Consequently, demand for all downstream segments will be metered by that bottleneck. The unsatisfied demand is stored in upstream segments, which causes queuing in Segment 7 and perhaps segments further upstream depending on the level of excess demand. The rate of growth of the vehicle queue (wave speed) is estimated from shock wave theory. The performance measures (speed and density) of any segment with queuing are recomputed, and the newly calculated values override the results from the segment-specific procedures.

Any unsatisfied demand is served in later time periods. As a result, volumes served in later time periods may be higher than the period demand flows. The resulting matrix of volumes served for Example Problem 2 is shown in Exhibit 25-56.

Exhibit 25-56
Example Problem 2:
Volume-Served Matrix

Time Step	Volumes Served (veh/h) by Segment										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	5,001	5,500	5,500	5,500	5,200	5,800	5,400	5,900	5,900	5,900	5,600
2	5,500	6,099	6,099	6,099	5,700	6,499	6,099	6,699	6,699	6,699	6,399
3	5,800	6,499	6,499	6,499	5,831	6,281	5,584	6,284	6,284	6,284	5,859
4	5,200	5,600	5,600	5,600	5,668	6,311	5,776	6,276	6,276	6,276	5,934
5	4,201	4,401	4,401	4,401	4,102	4,608	4,840	5,140	5,140	5,140	4,912

As a result of the bottleneck activation in Segment 8 in Time Period 3, queues form in upstream Segments 7, 6, and 5. The queuing is associated with reduced speeds and increased densities in those segments. The results in this chapter were obtained from the computational engine. The resulting performance measures computed for each segment and time interval are speed (Exhibit 25-57), density (Exhibit 25-58), and LOS (Exhibit 25-59).

Exhibit 25-57
Example Problem 2:
Speed Matrix

Time Step	Speed (mi/h) by Segment										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	59.8	53.2	58.6	55.9	59.5	46.8	59.0	52.5	52.5	55.7	58.3
2	58.6	52.1	55.8	55.5	57.9	45.4	55.8	50.6	50.6	51.5	53.9
3	57.4	51.1	53.1	53.1	45.3	24.2	28.1	51.6	51.6	54.7	57.1
4	47.2	47.5	51.5	48.3	56.5	24.7	29.6	51.7	51.7	54.7	56.8
5	60.0	54.5	59.7	56.2	60.0	51.4	50.9	53.7	53.7	56.1	59.9

Exhibit 25-58
Example Problem 2:
Density Matrix

Time Step	Density (veh/mi/ln) by Segment										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	27.9	34.5	31.3	32.8	29.2	31.0	30.5	37.4	37.4	35.3	32.0
2	31.3	39.0	36.4	36.7	32.8	35.8	36.4	44.2	44.2	43.3	39.6
3	33.7	42.4	40.8	40.8	42.9	64.8	66.4	40.6	40.6	38.3	34.2
4	36.7	39.3	36.3	38.6	33.4	63.9	65.1	40.4	40.4	38.2	34.8
5	23.3	26.9	24.5	26.1	22.8	22.4	31.7	31.9	31.9	30.5	27.3

Exhibit 25-59
Example Problem 2:
Expanded LOS Matrix

Time Step	Density-Based LOS by Segment										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	D	D	D	D	D	D	D	D	E	D	D
2	D	D	E	D	D	E	E	E	E	D	E
3	D	D	E	D	E	F	F	D	E	D	D
4	E	E	E	E	D	F	F	D	E	D	E
5	C	C	C	C	C	C	D	C	D	C	D
Time Step	**Demand-Based LOS by Segment**										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1											
2											
3								**F**	**F**	**F**	**F**
4											
5											

The LOS table for oversaturated facilities (Exhibit 25-59) distinguishes between the conventional density-based LOS and a segment demand-based LOS. The density-based stratification strictly depends on the prevailing average density on each segment. Segments downstream of the bottleneck, whose capacities are greater than or equal to the bottleneck capacity, operate at LOS E (or better), even though their v_d/c ratios are greater than 1.0. The demand-based LOS identifies those segments with demand-to-capacity ratios exceeding 1.0 as if they had been evaluated in isolation (i.e., using the methodologies of Chapters

12, 13, and 14). By contrasting the two parts of the LOS table, the analyst can develop an understanding of the metering effect of the bottleneck.

Step A-13: Apply Managed Lane Adjacent Friction Factor

This step is only required for facilities with managed lanes.

Step A-14: Compute Lane Group Performance

This step is only required for facilities with managed lanes.

Step A-15: Compute Freeway Facility Service Performance Measures by Time Interval

In the final analysis step, facilitywide performance measures are calculated for each time interval (Exhibit 25-60), consistent with Example Problem 1. Because the computations have already been shown, only summary results are shown here.

Exhibit 25-60
Example Problem 2: Facility Performance Measure Summary

Time Interval	Performance Measure: Space Mean Speed (mi/h)	Performance Measure: Average Density (veh/mi/ln)	LOS
1	56.8	31.0	D
2	54.4	36.2	E
3	42.5	45.6	F
4	42.5	43.8	E
5	56.4	26.2	D
Total	**50.5**	**35.6**	—

Step A-16: Aggregate to Section Level and Validate Against Field Data

This step validates the analysis and is performed only when field data are available.

Step A-17: Estimate LOS and Report Performance Measures for Lane Groups and Facility

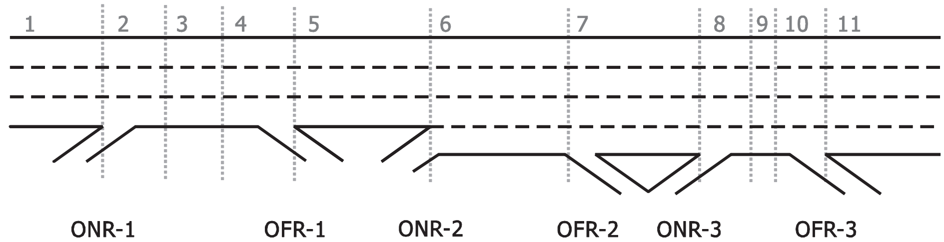
The LOS for each time interval is determined directly from the average density for each time interval. The facility operates at LOS F in Time Period 3 because one or more individual segments have demand-to-capacity ratios ≥ 1.0, even though the average facility density is below the LOS F threshold.

EXAMPLE PROBLEM 3: CAPACITY IMPROVEMENTS TO AN OVERSATURATED FACILITY

The Facility

In this example, portions of the congested facility in Example Problem 2 are being improved in an attempt to alleviate the congestion resulting from the Segment 8 bottleneck. Exhibit 25-61 shows the upgraded facility geometry.

Exhibit 25-61
Example Problem 3: Freeway Facility



The modified geometry of the 6-mi directional freeway facility is reflected in Exhibit 25-62.

Exhibit 25-62
Example Problem 3: Geometry of Directional Freeway Facility

Segment No.	1	2	3	4	5	6	7	8	9	10	11
Segment type	B	ONR	B	OFR	B	B or W	B	ONR	R	OFR	B
Segment length (ft)	5,280	1,500	2,280	1,500	5,280	2,640	5,280	1,140	360	1,140	5,280
No. of lanes	3	3	3	3	3	4	**4**	**4**	**4**	**4**	**4**

Notes: B = basic freeway segment; W = weaving segment; ONR = on-ramp (merge) segment; OFR = off-ramp (diverge) segment; R = overlapping ramp segment.
Bold type indicates geometry changes from Example Problems 1 and 2.

The facility improvements consisted of adding a lane to Segments 7–11 to give the facility a continuous four-lane cross section starting in Segment 6. The active bottleneck in Example Problem 2 was in Segment 8, but prior analysis showed that other segments (Segments 9–11) showed similar demand-to-capacity ratios greater than 1.0. Consequently, any capacity improvements that are limited to Segment 8 would have merely moved the spatial location of the bottleneck farther downstream rather than improving the overall facility. Segments 9–11 may also be referred to as "hidden" or "inactive" bottlenecks, because their predicted congestion is mitigated by the upstream metering of traffic.

The Facts

In addition to the information contained in Exhibit 25-61 and Exhibit 25-62, the following characteristics of the freeway facility are known:

SUTs and buses = 1.25% (all movements);

Mainline TTs = 1.00% (all movements);

Driver population = regular commuters;

FFS = 60 mi/h (all mainline segments);

Ramp *FFS* = 40 mi/h (all ramps);

Acceleration lane length = 500 ft (all ramps);

Deceleration lane length = 500 ft (all ramps);

D_{jam} = 190 pc/mi/ln;

c_{IFL} = 2,300 pc/h/ln (for *FFS* = 60 mi/h);

L_s = 1,640 ft (for Weaving Segment 6);

TRD = 1.0 ramp/mi;

Terrain = level;

Analysis duration = 75 min (divided into five 15-min intervals); and

Demand adjustment = +11% (all segments and all time intervals).

A queue discharge capacity drop of 7% is assumed.

Comments

The traffic demand flow inputs are identical to those in Example Problem 2, which reflected an 11% increase in traffic applied to all segments and all time periods relative to Example Problem 1. In an attempt to solve the congestion effect found in the earlier example, the facility was widened in Segments 7 through 11. This change directly affects the capacities of those segments.

In a more subtle way, the proposed modifications also change some of the defining parameters of Weaving Segment 6. With the added continuous lane downstream of the segment, the required number of lane changes from the ramp to the freeway is reduced from one to zero, following the guidelines in Chapter 13. These changes need to be considered when the undersaturated performance of that segment is evaluated. The weaving segment's capacity is unchanged relative to Example Problem 2 because, even with the proposed improvements, the number of weaving lanes remains two.

Step A-1: Define Study Scope

Similar to the previous example, the number of time steps is five and the facility has 11 segments. The analysis does not include any methodological extensions (i.e., managed lanes, reliability, ATDM, work zones).

Step A-2: Divide Facility into Sections and Segments

The segmentation of the subject facility is the same as in Example Problems 1 and 2 and is given in Exhibit 25-61. Therefore, the segmentation process is not repeated.

Step A-3: Input Data

Traffic demand inputs for all 11 segments and five analysis intervals are identical to those in Example Problem 2, as shown in Exhibit 25-53. The values represent the adjusted demand flows on the facility as determined from field observations or other sources. The actual volume served in each segment will be determined by using the methodologies and is expected to be less downstream of a congested segment. Additional geometric and traffic-related inputs are as specified in Exhibit 25-62 and the Facts section of the problem statement.

Step A-4: Balance Demands

The traffic flows in Exhibit 25-53 have already been given in the form of actual demands and no balancing is necessary.

Step A-5: Identify Global Parameters

Global inputs are jam density and queue discharge capacity drop. Values for both parameters are given in the Facts section of the problem statement.

Step A-6: Code Base Facility

In this step, all input data for the subject are coded in the computational engine.

Step A-7: Compute Segment Capacities

Segment capacities are determined by using the methodologies of Chapter 12 for basic freeway segments, Chapter 13 for weaving segments, and Chapter 14 for merge and diverge segments. The resulting capacities are shown in Exhibit 25-63. Because the capacity of a weaving segment depends on traffic patterns, it varies by time period. The remaining capacities are constant for all five time steps. The capacities for Segments 1–5 and Segments 7–11 are the same because the segments have the same basic cross section.

Exhibit 25-63
Example Problem 3:
Segment Capacities

Time Step	Capacities (veh/h) by Segment										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1						8,273					
2						8,281					
3	6,748	6,748	6,748	6,748	6,748	8,323	8,998	8,998	8,998	8,998	8,998
4						8,403					
5						8,463					

Step A-8: Calibrate with Adjustment Factors

This step allows the user to adjust demands, capacities, and FFSs for the purpose of calibration. There is no adjustment needed for the subject capacity according to the problem statement.

Step A-9: Adjust Managed Lane Cross Weave

This step is only required for facilities with managed lanes. The subject facility does not have a managed lane.

Step A-10: Compute Demand-to-Capacity Ratios

The demand-to-capacity ratios in Exhibit 25-64 are calculated from the demand flows in Exhibit 25-53 and segment capacities in Exhibit 25-63.

Exhibit 25-64
Example Problem 3:
Segment Demand-to-Capacity Ratios

Time Step	Demand-to-Capacity Ratio by Segment										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	0.74	0.82	0.82	0.82	0.77	0.70	0.60	0.66	0.66	0.66	0.62
2	0.82	0.90	0.90	0.90	0.84	0.78	0.68	0.74	0.74	0.74	0.71
3	0.86	0.96	0.96	0.96	0.92	0.85	0.74	0.82	0.82	0.82	0.77
4	0.77	0.83	0.83	0.83	0.79	0.68	0.59	0.64	0.64	0.64	0.61
5	0.62	0.65	0.65	0.65	0.61	0.52	0.47	0.50	0.50	0.50	0.48

The demand-to-capacity ratio matrix for Example Problem 3 (Exhibit 25-64) shows the capacity improvements successfully reduced all the previously congested segments to $v_d/c < 1.0$. Therefore, it is expected that the facility will operate as *globally undersaturated* and that all segment performance measures can be directly computed by using the methodologies in Chapters 12, 13, and 14.

Step A-11: Compute Undersaturated Segment Service Measures

Because the facility is globally undersaturated, the methodology proceeds to calculate service measures for each segment and each time period, starting with the first segment in Time Step 1. The computational details for each segment type are exactly as described in Chapters 12, 13, and 14. The basic performance service measures computed for each segment and each time interval include segment speed (Exhibit 25-65), density (Exhibit 25-66), and LOS (Exhibit 25-67).

Exhibit 25-65
Example Problem 3:
Speed Matrix

Time Step	Speed (mi/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	59.8	53.2	58.6	55.9	59.5	50.5	60.0	54.9	54.9	58.1	60.0
2	58.6	52.1	55.8	55.5	57.9	50.1	60.0	54.3	54.3	57.7	60.0
3	57.4	51.1	53.1	53.1	55.2	49.7	59.8	53.6	53.6	57.2	59.5
4	59.5	53.0	58.3	55.8	59.2	50.8	60.0	55.0	55.0	58.1	60.0
5	60.0	54.5	59.7	56.2	60.0	53.4	60.0	55.9	55.9	58.8	60.0

Exhibit 25-66
Example Problem 3:
Density Matrix

Time Step	Density (veh/mi/ln) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	27.9	34.5	31.3	32.8	29.2	28.7	22.5	26.8	26.8	25.4	23.3
2	31.3	39.0	36.4	36.7	32.8	32.5	25.4	30.9	30.9	29.0	26.7
3	33.7	42.4	40.8	40.8	37.4	35.7	28.0	34.5	34.5	32.4	29.0
4	29.2	35.2	32.0	33.4	29.8	28.1	22.1	26.4	26.4	24.9	22.9
5	23.3	26.9	24.5	26.1	22.8	20.6	17.5	20.1	20.1	19.1	17.9

Exhibit 25-67
Example Problem 3:
LOS Matrix

Time Step	LOS by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	D	D	D	D	D	D	C	C	D	C	C
2	D	D	E	D	D	D	C	C	D	C	D
3	D	D	E	D	E	E	D	D	D	D	D
4	D	D	D	D	D	D	C	C	D	C	C
5	C	C	C	C	C	C	B	B	C	B	B

Step A-13: Apply Managed Lane Adjacent Friction Factor

This step is only required for facilities with managed lanes.

Step A-14: Compute Lane Group Performance

This step is only required for facilities with managed lanes.

Step A-15: Compute Freeway Facility Service Performance Measures by Time Interval

In this analysis step, facilitywide performance measures are calculated for each time step (Exhibit 25-68), consistent with Example Problem 2. Because the computations have already been shown, only summary results are shown here. The improvement restored the facility LOS to the values experienced in the original pregrowth scenario, as shown in Exhibit 25-68.

Exhibit 25-68
Example Problem 3: Facility Performance Measure Summary

Time Step	Performance Measure: Space Mean Speed (mi/h)	Performance Measure: Average Density (veh/mi/ln)	LOS
1	57.9	26.8	D
2	57.1	30.3	D
3	55.9	33.5	D
4	57.8	26.9	D
5	58.6	20.8	C
Total	**57.5**	**27.7**	**—**

Step A-16: Aggregate to Section Level and Validate Against Field Data

This step validates the analysis and is performed only when field data are available.

Step A-17: Estimate LOS and Report Performance Measures for Lane Groups and Facility

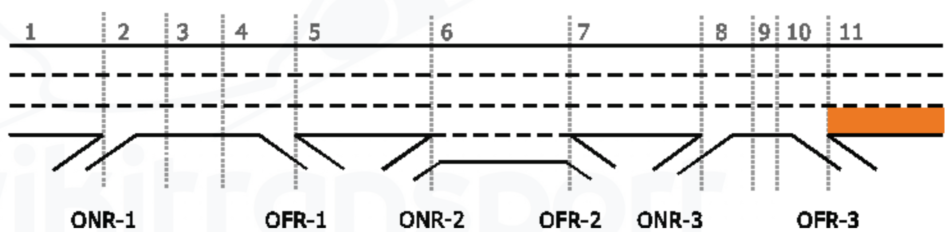
The LOS for each time interval is determined directly from the average density for each time interval. The improvement restored the facility LOS to the values experienced in the original pregrowth (undersaturated) scenario shown in Exhibit 25-51.

EXAMPLE PROBLEM 4: EVALUATION OF AN UNDERSATURATED FACILITY WITH A WORK ZONE

The Facility

In this example, a long-term work zone is placed on the final segment of Example Problem 1. Exhibit 25-69 shows the change to the facility.

Exhibit 25-69
Example Problem 4: Freeway Facility



The modified geometry of the 6-mi directional freeway facility is reflected in Exhibit 25-70.

Exhibit 25-70
Example Problem 4: Geometry of Directional Freeway Facility

Segment No.	1	2	3	4	5	6	7	8	9	10	11
Segment type	B	ONR	B	OFR	B	B or W	B	ONR	R	OFR	B
Segment length (ft)	5,280	1,500	2,280	1,500	5,280	2,640	5,280	1,140	360	1,140	5,280
No. of lanes	3	3	3	3	3	4	3	3	3	3	2

Notes: B = basic freeway segment; W = weaving segment; ONR = on-ramp (merge) segment; OFR = off-ramp (diverge) segment; R = overlapping ramp segment.

The Facts

In addition to the information contained in Exhibit 25-69 and Exhibit 25-70, the following characteristics of the freeway facility are known:

SUTs and buses = 1.25% (all movements);

Mainline TTs = 1.00% (all movements);

Driver population = regular commuters;

FFS = 60 mi/h (all mainline segments);

Ramp *FFS* = 40 mi/h (all ramps);

Acceleration lane length = 500 ft (all ramps);

Deceleration lane length = 500 ft (all ramps);

D_{jam} = 190 pc/mi/ln;

c_{IFL} = 2,300 pc/h/ln (for *FFS* = 60 mi/h);

L_s = 1,640 ft (for Weaving Segment 6);

TRD = 1.0 ramp/mi;

Terrain = level; and

Analysis duration = 75 min (divided into five 15-min intervals).

A queue discharge capacity drop of 7% is assumed for non–work zone conditions.

Comments

The traffic demand flow inputs are identical to those in Example Problem 1. The work zone has a single lane closure (in Segment 11), concrete barriers, and a lateral distance of 12 ft in an urban area. Daytime performance is of interest throughout the analysis.

Step A-1: Define Study Scope

Similar to the previous examples, there are five time steps and the facility has 11 segments. The work zone extension to the methodology will be included as part of the analysis.

Step A-2: Divide Facility into Sections and Segments

The segmentation of the subject facility is given in Exhibit 25-69. Therefore, there is no need to go through the segmentation process.

Step A-3: Input Data

Traffic demand inputs for all 11 segments and five analysis intervals are identical to those in Example Problem 1, as shown in Exhibit 25-45. The values represent the adjusted demand flows on the facility as determined from field observations or other sources. Additional geometric and traffic-related inputs are as specified in Exhibit 25-70 and the Facts section of the problem statement.

Step A-4: Balance Demands

The traffic flows in Exhibit 25-45 have already been given in the form of actual demands and no balancing is necessary.

Step A-5: Identify Global Parameters

Global inputs are jam density and queue discharge capacity drop. Values for both parameters are given in the Facts section of the problem statement.

Step A-6: Code Base Facility

In this step, all input data for the subject facility are coded in the computational engine.

Step A-7: Compute Segment Capacities

The resulting capacities are shown in Exhibit 25-71. Because the capacity of a weaving segment depends on traffic patterns, it varies by time period. The remaining capacities are constant for all five time steps. The capacities for Segments 1–5 and for Segments 7–10 are the same because the segments have the same basic cross section. The lane closure on Segment 11 reduces its base capacity by 33%. The impacts of work zone presence on further capacity reduction are assessed in the next step.

Exhibit 25-71
Example Problem 4:
Segment Capacities

Time Step	Capacities (veh/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1						8,273					
2						8,281					
3	6,748	6,748	6,748	6,748	6,748	8,323	6,748	6,748	6,748	6,748	4,499
4						8,403					
5						8,463					

Step A-8: Calibrate with Adjustment Factors

To calculate the CAF for the work zone (Segment 11), the queue discharge and prebreakdown capacities are required. As a result of the work zone, Segment 11 has two open lanes and one closed lane. Therefore, from Exhibit 10-15, its lane closure severity index $LCSI$ value is equal to 0.75. Equation 10-8 gives the segment's queue discharge capacity as follows:

$$QDR_{wz} = 2{,}093 - 154 \times LCSI - 194 \times f_{Br} - 179 \times f_{AT} + 9 \times f_{LAT} - 59 \times f_{DN}$$

$$\begin{aligned} QDR_{wz} &= 2{,}093 - 154 \times LCSI - 194 \times f_{Br} - 179 \times f_{AT} + 9 \times f_{LAT} - 59 \times f_{DN} \\ &= 2{,}093 - 154 \times 0.75 - 194 \times 0 - 179 \times 0 - 59 \times 0 + 9 \times 0 \\ &= 1{,}977.5 \text{ veh/h} \end{aligned}$$

Using Equation 10-9 and assuming a 13.1% queue discharge capacity drop in work zone conditions, prebreakdown capacity is calculated as follows:

$$c_{WZ} = \frac{QDR_{wz}}{100 - \alpha_{wz}} \times 100$$

$$c_{WZ} = \frac{1{,}977.5}{100 - 13.1} \times 100$$

$$c_{WZ} = 2{,}275.6 \text{ veh/h}$$

Then, from Equation 10-11, the work zone CAF is equal to

$$CAF_{wz} = \frac{c_{wz}}{c} = \frac{2{,}275.6}{2{,}300} = 0.989$$

Using a similar approach, the work zone SAF can be found as follows from Equation 10-10 and Equation 10-12.

$$FFS_{wz} = 9.95 + 33.49 \times f_{Sr} + 0.53 \times SL_{wz} - 5.60 \times LCSI - 3.84 \times f_{Br}$$
$$-1.71 \times f_{DN} - 8.7 \times TRD$$

$$FFS_{wz} = 9.95 + 33.49 \times \left(\frac{60}{55}\right) + 0.53 \times 55 - 5.60 \times 0.75 - 3.84 \times 1$$
$$-1.71 \times 0 - 8.7 \times 1$$

$$FFS_{wz} = 58.9 \text{ mi/h}$$

$$SAF_{wz} = \frac{FFS_{wz}}{FFS} = \frac{58.9}{60} = 0.982$$

These values will be used to update the capacity and FFS of Segment 11 in all time intervals. In addition, the number of lanes in the segment will be reduced to two.

Step A-9: Adjust Managed Lane Cross Weave

This step is only required for facilities with managed lanes. The subject facility does not have a managed lane.

Step A-10: Compute Demand-to-Capacity Ratios

The demand-to-capacity ratios shown in Exhibit 25-72 are calculated from the demand flows in Exhibit 25-45 and segment capacities in Exhibit 25-71.

Exhibit 25-72
Example Problem 4: Segment Demand-to-Capacity Ratios

Time Step	Demand-to-Capacity Ratio by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	0.67	0.73	0.73	0.73	0.69	0.63	0.72	0.79	0.79	0.79	1.26
2	0.73	0.81	0.81	0.81	0.76	0.71	0.81	0.89	0.89	0.89	1.44
3	0.77	0.87	0.87	0.87	0.83	0.77	0.89	0.99	0.99	0.99	1.56
4	0.69	0.75	0.75	0.75	0.71	0.61	0.71	0.77	0.77	0.77	1.24
5	0.56	0.59	0.59	0.59	0.55	0.47	0.56	0.60	0.60	0.60	0.97

The demand-to-capacity ratio matrix for Example Problem 4 (Exhibit 25-72) shows the presence of the work zone significantly increases the demand-to-capacity ratio on Segment 11. Queues are very likely to start to grow and spill back to upstream segments, and the facility is expected to operate in oversaturated conditions.

Step A-12: Compute Oversaturated Segment Service Measures

The computations for oversaturation apply to any segment with a v_d/c ratio greater than 1.0, as well as any segments upstream of those segments that experience queuing as a result of the bottleneck. All remaining segments are analyzed by using the individual segment methodologies of Chapters 12, 13, and 14, as applicable, with the caveat that the volumes served may differ from the demand flows.

Similar to Example Problem 1, in Example Problem 4, the methodology calculates performance measures for each segment and each time period, starting with the first segment in Time Step 1. The computations are repeated for the first 10 segments for Time Step 1 without encountering a segment with $v_d/c > 1.0$. Once the methodology enters Segment 11 in Time Step 1, the oversaturated computational module is invoked.

The v_a/c ratio for Segment 11, which has the first active bottleneck, will be more than 1.0 and the segment will process traffic at its capacity. Consequently, demand for all downstream segments will be metered by that bottleneck. The unsatisfied demand is stored in upstream segments, which causes queuing in Segment 10 and perhaps additional upstream segments, depending on the level of excess demand. The rate of growth of the vehicle queue (wave speed) is estimated from shock wave theory. The performance measures (speed and density) of any segment with queuing are recomputed, and the newly calculated values override the results from the segment-specific procedures.

Any unsatisfied demand is served in later time periods. As a result, volumes served in later time periods may be higher than the period demand flows. The resulting matrix of volumes served for Example Problem 4 is shown in Exhibit 25-73.

Exhibit 25-73
Example Problem 4:
Volume-Served Matrix

Time Step	Volumes Served (veh/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	4,505	4,955	4,955	4,955	4,685	5,225	3,924	4,185	4,126	3,929	3,719
2	4,955	5,495	5,495	5,446	3,947	3,701	3,325	3,878	3,882	3,895	3,714
3	3,275	3,476	3,094	3,031	2,912	3,391	3,250	3,899	3,905	3,929	3,714
4	2,831	3,398	3,474	3,416	3,424	3,914	3,597	4,014	4,004	3,965	3,714
5	3,589	3,991	4,096	3,957	3,452	3,912	3,675	3,923	3,916	3,897	3,714

As a result of the bottleneck activation (due to the work zone's presence) in Segment 11 in Time Step 1, queues form in upstream Segments 10, 9, 8, 7, and 6. The queuing is associated with reduced speeds and increased densities in those segments. These and subsequent results were obtained from the computational engine. The resulting performance measures computed for each segment and time interval are speed (Exhibit 25-74), density (Exhibit 25-75), and LOS (Exhibit 25-76). Similar trends are observed in the following time intervals, with queueing reaching the beginning of the facility.

Exhibit 25-74
Example Problem 4:
Speed Matrix

Time Step	Speed (mi/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	60.0	53.9	59.7	56.1	60.0	48.0	24.2	15.9	13.0	13.0	50.4
2	59.9	53.2	54.5	52.3	22.2	8.9	9.4	12.3	12.2	12.2	50.5
3	12.9	12.8	13.1	9.7	8.0	6.5	9.1	12.4	12.4	12.4	50.5
4	5.9	11.0	12.9	12.8	11.5	8.3	11.0	13.1	12.7	12.7	50.5
5	11.0	16.4	18.6	16.4	12.3	8.3	11.2	12.5	12.3	12.3	50.5

Exhibit 25-75
Example Problem 4:
Density Matrix

Time Step	Density (veh/mi/ln) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	25.0	30.6	27.6	29.4	26.0	27.2	54.1	87.5	100.6	100.6	36.9
2	27.6	34.5	33.6	34.7	59.1	104.2	117.8	105.5	106.2	106.2	36.8
3	84.6	90.6	78.7	104.6	121.4	130.1	119.1	104.4	105.4	105.4	36.8
4	159.3	103.4	89.8	88.7	99.4	117.3	109.0	102.5	104.2	104.2	36.8
5	108.6	81.0	73.5	80.4	93.5	118.2	109.2	105.0	106.0	106.0	36.8

Exhibit 25-76
Example Problem 4:
LOS Matrix

Time Step	LOS by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	C	C	D	C	D	C	F	F	F	F	E
2	D	D	D	D	F	F	F	F	F	F	E
3	F	F	F	F	F	F	F	F	F	F	E
4	F	F	F	F	F	F	F	F	F	F	E
5	F	F	F	F	F	F	F	F	F	F	E

Step A-13: Apply Managed Lane Adjacent Friction Factor

This step is only required for facilities with managed lanes.

Step A-14: Compute Lane Group Performance

This step is only required for facilities with managed lanes.

Step A-15: Compute Freeway Facility Service Performance Measures by Time Interval

In the final analysis step, facilitywide performance measures are calculated for each time step (Exhibit 25-77). Because the computations have already been demonstrated in previous example problems, only summary results are shown. The work zone presence created significant congestion on the subject facility.

Exhibit 25-77
Example Problem 4:
Facility Performance Measure Summary

Time Step	Performance Measure: Space Mean Speed (mi/h)	Performance Measure: Average Density (veh/mi/ln)	LOS
1	39.2	38.4	F
2	21.8	66.1	F
3	11.5	99.1	F
4	11.3	105.5	F
5	13.7	93.4	F
Total	19.5	80.5	—

Step A-16: Aggregate to Section Level and Validate Against Field Data

This step validates the analysis and is performed only when field data are available.

Step A-17: Estimate LOS and Report Performance Measures for Lane Groups and Facility

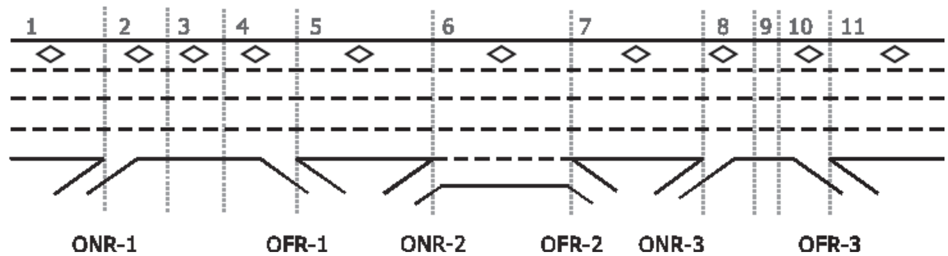
The LOS for each time interval is determined directly from the average density for each time interval. Work zone presence eroded the facility LOS to F in all time intervals.

EXAMPLE PROBLEM 5: EVALUATION OF AN OVERSATURATED FACILITY WITH A MANAGED LANE

The Facility

In this example, a managed lane will be added to the freeway facility described in Example Problem 2. Exhibit 25-78 shows the new facility geometry.

Exhibit 25-78
Example Problem 5: Freeway Facility



Details of the modified geometry of the 6-mi directional freeway facility are provided in Exhibit 25-79.

Exhibit 25-79
Example Problem 5: Geometry of Directional Freeway Facility

Segment No.	1	2	3	4	5	6	7	8	9	10	11
Segment type	B	ONR	B	OFR	B	B or W	B	ONR	R	OFR	B
Segment length (ft)	5,280	1,500	2,280	1,500	5,280	2,640	5,280	1,140	360	1,140	5,280
No. of GP lanes	3	3	3	3	3	4	3	3	3	3	3
No. of ML	1	1	1	1	1	1	1	1	1	1	1

Notes: B = basic freeway segment; W = weaving segment; ONR = on-ramp (merge) segment; OFR = off-ramp (diverge) segment; R = overlapping ramp segment; GP = general purpose; ML = managed lanes.

The Facts

In addition to the information contained in Exhibit 25-78 and Exhibit 25-79, the following characteristics of the freeway facility are known:

SUTs and buses = 1.25% (all movements);

Mainline TTs = 1.00% (all movements);

Driver population = regular commuters;

FFS = 60 mi/h (all mainline segments);

Ramp *FFS* = 40 mi/h (all ramps);

Acceleration lane length = 500 ft (all ramps);

Deceleration lane length = 500 ft (all ramps);

D_{jam} = 190 pc/mi/ln;

c_{IFL} = 2,300 pc/h/ln (for *FFS* = 60 mi/h);

L_s = 1,640 ft (for Weaving Segment 6);

TRD = 1.0 ramp/mi;

Terrain = level;

Analysis duration = 75 min (divided into five 15-min intervals); and

Demand adjustment = +11% (all segments and all time intervals).

A queue discharge capacity drop of 7% is assumed.

Comments

The traffic demand flow inputs are identical to those in Example Problem 2. The facility includes a single managed lane separated with marking with FFS equal to 60 mi/h. The lane is a basic managed lane with no intermediate access points. It is assumed 20% of entry traffic demand on the mainline will use the managed lane.

Step A-1: Define Study Scope

Similar to the previous examples, there are five time steps and the facility has 11 segments. The managed lane extension to the methodology will be used for this analysis.

Step A-2: Divide Facility into Sections and Segments

The segmentation of the subject facility is given in Exhibit 25-78. Therefore, the segmentation process is not repeated.

Step A-3: Input Data

On- and off-ramp demand flow rates are identical to those of Example Problem 2, shown in Exhibit 25-53. It is assumed total entry volume is identical to that of Example Problem 2; however, 20% of total demand is allocated to the managed lane, and the remaining 80% to the general purpose lanes, as shown in Exhibit 25-80.

Exhibit 25-80
Example Problem 5: Demand Inputs on the Mainline

Time Step	Entering Flow Rate on General Purpose Lanes (veh/h)	Entering Flow Rate on Managed Lane (veh/h)	Sum of Entering Flow Rate to the Facility (veh/h)
1	4,001	1,000	5,001
2	4,400	1,100	5,500
3	4,640	1,160	5,800
4	4,160	1,040	5,200
5	3,361	840	4,201

Step A-4: Balance Demands

The traffic flows in Exhibit 25-53 and Exhibit 25-80 have already been given in the form of actual demands and no balancing is necessary.

Step A-5: Identify Global Parameters

Global inputs are jam density and queue discharge capacity drop. Values for both parameters are given in the problem statement.

Step A-6: Code Base Facility

In this step, all input data for the subject facility are coded in the computational engine.

Step A-7: Compute Segment Capacities

Segment capacities are determined by using the methodologies of Chapter 12 for basic freeway segments (general purpose and managed lanes), Chapter 13 for weaving segments, and Chapter 14 for merge and diverge segments. The resulting capacities are shown in Exhibit 25-81.

Exhibit 25-81
Example Problem 5: Segment Capacities

Time Step	Capacities (veh/h) by Segment for General Purpose Lanes										
	1	2	3	4	5	6	7	8	9	10	11
1						8,177					
2						8,189					
3	6,748	6,748	6,748	6,748	6,748	8,244	6,748	6,748	6,748	6,748	6,748
4						8,331					
5						8,403					

Time Step	Capacities (veh/h) by Segment for Managed Lane										
	1	2	3	4	5	6	7	8	9	10	11
1											
2											
3	1,614	1,614	1,614	1,614	1,614	1,614	1,614	1,614	1,614	1,614	1,614
4											
5											

Step A-8: Calibrate with Adjustment Factors

This step allows the analyst to adjust demands, capacities, and FFSs for the purpose of calibration. According to the problem statement, there is no adjustment needed for the subject facility's capacity.

Step A-9: Adjust Managed Lane Cross Weave

This facility does not have a cross weave. Therefore, this step is skipped.

Step A-10: Compute Demand-to-Capacity Ratios

The demand-to-capacity ratios shown in Exhibit 25-82 are calculated from the demand flows in Exhibit 25-53 and Exhibit 25-80 and segment capacities in Exhibit 25-81.

Exhibit 25-82
Example Problem 5: Segment Demand-to-Capacity Ratios

Time Step	Demand-to-Capacity Ratio by Segment (General Purpose Lanes)										
	1	2	3	4	5	6	7	8	9	10	11
1	0.59	0.67	0.67	0.67	0.62	0.59	0.65	0.73	0.73	0.73	0.68
2	0.65	0.74	0.74	0.74	0.68	0.66	0.74	0.83	0.83	0.83	0.79
3	0.69	0.79	0.79	0.79	0.75	0.72	0.82	0.92	0.92	0.92	0.85
4	0.62	0.68	0.68	0.68	0.63	0.56	0.63	0.71	0.71	0.71	0.66
5	0.50	0.53	0.53	0.53	0.48	0.42	0.50	0.54	0.54	0.54	0.51

Time Step	Demand-to-Capacity Ratio by Segment (Managed Lane)										
	1	2	3	4	5	6	7	8	9	10	11
1	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62
2	0.68	0.68	0.68	0.68	0.68	0.68	0.68	0.68	0.68	0.68	0.68
3	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
4	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64
5	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.52

The demand-to-capacity ratio matrix for Example Problem 5 (Exhibit 25-82) shows the addition of the managed lane improves traffic operations on the general purpose lanes. As such, it is expected the facility will operate in undersaturated conditions.

Step A-11: Compute Undersaturated Segment Service Measures

The computations for oversaturation apply to any segment with a v_d/c ratio greater than 1.0 as well as any segments upstream of those segments that experience queuing as a result of the bottleneck. All remaining segments are analyzed by using the individual segment methodologies of Chapters 12, 13, and

14, as applicable, with the caveat that volumes served may differ from demand flows.

The basic performance service measures computed for each segment and each time interval include segment speed (Exhibit 25-83), density (Exhibit 25-84), and LOS (Exhibit 25-85).

Exhibit 25-83
Example Problem 5: Speed Matrix

Time Step	Speed (mi/h) by Segment (General Purpose Lanes)										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	60.0	54.4	59.7	56.2	60.0	48.0	60.0	54.0	54.0	56.1	60.0
2	60.0	53.8	59.7	55.9	60.0	46.8	59.8	53.0	53.0	55.8	59.2
3	60.0	53.3	59.1	55.9	59.7	46.2	58.5	51.7	51.7	55.0	57.7
4	60.0	54.3	59.7	56.2	60.0	49.9	60.0	54.1	54.1	56.1	60.0
5	60.0	55.2	59.8	56.3	60.0	52.7	60.0	55.1	55.1	56.5	60.0
Time Step	**Speed (mi/h) by Segment (Managed Lane)**										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	59.3	59.3	59.3	59.3	59.3	59.3	59.3	59.3	59.3	59.3	59.3
2	58.9	58.9	58.9	58.9	58.9	58.9	58.9	53.5	53.5	58.1	58.9
3	58.6	58.6	58.6	58.6	58.6	58.6	58.6	52.1	52.1	52.1	58.6
4	59.2	59.2	59.2	59.2	59.2	59.2	59.2	59.2	59.2	59.2	59.2
5	59.7	59.7	59.7	59.7	59.7	59.7	59.7	59.7	59.7	59.7	59.7

Exhibit 25-84
Example Problem 5: Density Matrix

Time Step	Density (veh/mi/ln) by Segment (General Purpose Lanes)										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	22.2	27.6	25.0	26.7	23.3	25.0	24.4	30.3	30.3	29.1	25.6
2	24.4	31.0	27.9	29.8	25.6	28.9	27.9	35.2	35.2	33.4	29.8
3	25.8	33.4	30.1	31.8	28.1	32.2	31.6	40.2	40.2	37.8	33.2
4	23.1	28.0	25.3	27.1	23.7	23.4	23.7	29.3	29.3	28.3	24.8
5	18.7	21.5	19.8	21.1	18.1	16.9	18.7	22.1	22.1	21.6	19.2
Time Step	**Density (veh/mi/ln) by Segment (Managed Lane)**										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	16.9	16.9	16.9	16.9	16.9	16.9	16.9	16.9	16.9	16.9	16.9
2	18.7	18.7	18.7	18.7	18.7	18.7	18.7	20.6	20.6	18.7	18.7
3	19.8	19.8	19.8	19.8	19.8	19.8	19.8	22.3	22.3	22.3	19.8
4	17.6	17.6	17.6	17.6	17.6	17.6	17.6	17.6	17.6	17.6	17.6
5	14.1	14.1	14.1	14.1	14.1	14.1	14.1	14.1	14.1	14.1	14.1

Exhibit 25-85
Example Problem 5: LOS Matrix

Time Step	LOS by Segment (General Purpose Lanes)										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	C	C	C	C	C	C	C	C	D	C	C
2	C	C	D	C	C	D	D	D	E	D	D
3	C	D	D	D	D	D	D	D	E	D	D
4	C	C	C	C	C	C	C	C	D	C	C
5	C	B	C	C	C	B	C	B	C	C	C
Time Step	**LOS by Segment (Managed Lane)**										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	B	B	B	B	B	B	B	B	B	B	B
2	C	C	C	C	C	C	C	C	C	C	C
3	C	C	C	C	C	C	C	C	C	C	C
4	B	B	B	B	B	B	B	B	B	B	B
5	B	B	B	B	B	B	B	B	B	B	B

Step A-13: Apply Managed Lane Adjacent Friction Factor

The subject facility has densities in excess of 35 pc/mi/ln. As a result, friction effects are applied according to the process described in Chapter 12. The indicator variable I_c in Equation 12-12 will have a nonzero value for the segments and analysis periods during which the general purpose lane density is greater than 35 pc/mi/ln. Consequently, the S_3 term in Equation 12-12 will reduce the estimated general purpose lane speed as a result of the friction.

Step A-14: Compute Lane Group Performance

In this step, performance measures for all the facility's lane groups are computed. The subject facility has two lane groups, one for general purpose lanes and one for the managed lane, as shown in Exhibit 25-86.

Exhibit 25-86
Example Problem 5: Facility Performance Measure Summary for Lane Groups

Time Step	General Purpose Lane Group Performance Measure		Managed Lane Group Performance Measure	
	Space Mean Speed (mi/h)	Average Density (veh/mi/ln)	Space Mean Speed (mi/h)	Average Density (veh/mi/ln)
1	57.7	24.9	59.3	16.9
2	57.3	28.1	58.6	18.8
3	56.5	31.0	58.0	20.0
4	58.0	24.6	59.2	17.6
5	58.5	19.1	59.7	14.1

Step A-15: Compute Freeway Facility Service Performance Measures by Time Interval

In the final analysis step, facilitywide performance measures are calculated for each time step (Exhibit 25-87). Because the computations have been demonstrated previously, only summary results are shown here. The addition of the managed lane reduced traffic congestion on the subject facility.

Exhibit 25-87
Example Problem 5: Facility Performance Measure Summary

Time Step	Performance Measure		LOS
	Space Mean Speed (mi/h)	Average Density (veh/mi/ln)	
1	58.0	23.4	C
2	57.5	26.4	D
3	56.7	29.1	D
4	58.2	23.3	C
5	58.7	18.1	C
Total	57.8	24.0	—

Step A-16: Aggregate to Section Level and Validate Against Field Data

This step validates the analysis and is performed only when field data are available.

Step A-17: Estimate LOS and Report Performance Measures for Lane Groups and Facility

The LOS for each time interval is determined directly from the average density for each time interval. The addition of the managed lane improved traffic conditions over the entire facility.

EXAMPLE PROBLEM 6: PLANNING-LEVEL ANALYSIS OF A FREEWAY FACILITY

The Facility

In this example, the planning-level methodology is used to analyze a freeway facility with geometric characteristics identical to the facility used in Example Problem 1. Exhibit 25-43 shows the facility geometry. Note that the planning methodology uses annual average daily traffic (AADT) values to calculate demand levels at the facility's entry and exit points based on the hourly (K) and annual growth factors (f_g). As a result, although the AADTs have been manipulated in this example to create demand levels close to those of Example Problem 1, the results will not match precisely. Furthermore, because the planning-level methodology uses freeway sections rather than segments and is limited to four analysis periods, a direct comparison is not possible.

The Facts

In addition to the information given in Exhibit 25-43 and Exhibit 25-44, the following characteristics of the freeway facility are known:

Heavy-vehicle percentage = 0%,

Driver population = regular commuters on an urban facility,

FFS = 60 mi/h (all mainline segments),

Ramp *FFS* = 40 mi/h (all ramps),

D_{jam} = 190 pc/mi/ln,

K-factor = 0.09,

Growth factor = 1,

PHF = 0.9,

Terrain = level, and

Analysis duration = 60 min (divided into four 15-min analysis periods).

Average Annual Daily Traffic

The planning-level approach uses directional AADT values to approximate demand levels on different freeway sections. Exhibit 25-88 depicts AADT values on all entry points (i.e., the first basic freeway section and all on-ramps) and all exit points (all off-ramps).

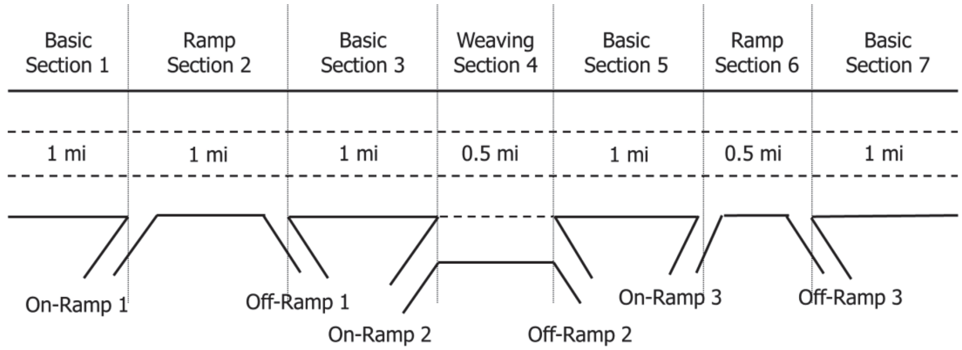
Exhibit 25-88
Example Problem 6:
AADT Values for the Facility

Entering AADT (veh/day)	Ramp AADT (veh/day)					
	ONR1	ONR2	ONR3	OFR1	OFR2	OFR3
55,000	4,500	5,400	4,500	2,700	3,600	2,700

Sections

The facility and all geometric inputs are identical to Example Problem 1. Exhibit 25-89 presents the different freeway sections for the facility of interest.

Exhibit 25-89
Example Problem 6: Section Definition for the Facility



Section 1 is a basic section, identical to the HCM segmentation definition. An on-ramp roadway is located just downstream of Section 1 that results in changes in the demand level. As a result, a new section needs to be defined. The demand level on the new section remains fixed up to the first off-ramp roadway, at which point both the capacity and the demand change. As a result, Section 2 is defined as a ramp section. After the off-ramp roadway, the facility demand drops and remains fixed until the next on-ramp roadway. As a result, Section 3 is defined as a basic freeway section. Sections on the rest of the freeway facility are defined following a similar process. The result is that seven distinct sections are defined.

Step 1: Demand Level Calculations

The demand level on each section in each analysis period is determined by using the given AADT values, *PHF*, *K*-factor, heavy-vehicle factor, and growth factor.

$$q_{1,1} = AADT_1 \times K \times f_{tg} \times f_{HV} = 55{,}000 \times 0.09 \times 1 \ \times 1 \ = 4{,}950 \text{ pc/h}$$

$$q_{1,2} = AADT_1 \times K \times \left(\frac{1}{PHF}\right) \times f_{tg} \times f_{HV} = 55{,}000 \times 0.09 \times \left(\frac{1}{0.9}\right) \times 1$$
$$= 5{,}500 \text{ pc/h}$$

$$q_{1,3} = AADT_1 \times K \times f_{tg} = 55{,}000 \times 0.09 \times 1 \times f_{HV} = 4{,}950 \text{ pc/h}$$

$$q_{1,4} = AADT_1 \times K \times \left(2 - \frac{1}{PHF}\right) \times f_{tg} \times f_{HV} = 55{,}000 \times 0.09 \times \left(2 - \frac{1}{0.9}\right) \times 1$$
$$= 4{,}400 \text{ pc/h}$$

By following the same approach, the demand levels for all facility entry and exit points are found. The results are summarized in Exhibit 25-90.

Exhibit 25-90
Example Problem 6: Demand Flow Rates (pc/h) on the Subject Facility

Analysis Period	Entry	On-Ramp 1	Off-Ramp 1	On-Ramp 2	Off-Ramp 2	On-Ramp 3	Off-Ramp 3
1	4,950	405	243	486	324	405	243
2	5,500	450	270	540	360	450	270
3	4,950	405	243	486	324	405	243
4	4,400	360	216	432	288	360	216

After calculation of the entry and exit demand flow rates from the AADT values, the demand level in each section in each analysis period is found.

Step 2: Section Capacity Calculations and Adjustments

Equation 25-45 is used to determine the base capacity of each section. The base capacity of each section is then adjusted by using the appropriate adjustment factor for a weaving, ramp, merge, or diverge section. For instance, the capacity of Section 1 (a basic section) is determined as follows:

$$c_1 = \left(2{,}200 + 10 \times (\min(70, S_{FFS}) - 50)\right) = \left(2{,}200 + 10 \times (\min(70, 60) - 50)\right)$$

$$c_1 = 2{,}300 \text{ pc/h/ln}$$

Because FFS and percentage heavy vehicles are global inputs, the capacity of each of the facility's basic freeway sections is equal to 2,300 pc/h/ln. However, for all other sections, this base capacity needs to be adjusted.

Section 2 is a ramp section. The CAF for a ramp section is 0.9. Therefore, the capacity of Section 2 is computed as follows:

$$c_2 = 2300 \times 0.90 = 2{,}070 \text{ pc/h/ln}$$

Section 3 is a basic freeway section; therefore, its capacity remains at 2,300 pc/h/ln. However, Section 4 is a weaving section and its capacity will need to be adjusted. The CAF for a weaving section is determined by the volume ratio and section length.

The volume ratio (the ratio of weaving demand to total demand) is approximated by summing the weaving section's ramp AADT values and dividing the result by the total AADT on the weaving section, as follows:

$$V_r = \frac{(5{,}400 + 3{,}600)}{55{,}000 + 4{,}500 - 2{,}700} = \frac{9{,}000}{56{,}800} = 0.158$$

The length of the weaving section is 0.5 mi. As a result, the CAF is calculated as follows:

$$CAF_{\text{weave}} = \min(0.884 - 0.0752 V_r + 0.0000243 L_s, 1)$$

$$CAF_{\text{weave}} = 0.884 - 0.0752 \times 0.164 + 0.0000243 \times 0.5 \times 5{,}280 = 0.94$$

Therefore, the capacity of Section 4 is

$$c_4 = 2{,}300 \times 0.94 = 2{,}162 \text{ pc/h/ln}$$

The capacities of Section 5 (basic), Section 6 (ramp), and Section 7 (basic) are 2,300, 2,070, and 2,300 pc/h/ln, respectively. At this stage, demand-to-capacity ratios for all sections in all analysis periods can be determined, as presented in Exhibit 25-91.

Exhibit 25-91
Example Problem 6: Demand-to-Capacity Ratios by Section and Analysis Period

Analysis Period	Demand-to-Capacity Ratios by Section						
	1	2	3	4	5	6	7
1	0.72	0.86	0.74	0.65	0.76	0.91	0.79
2	0.80	0.96	0.82	0.72	0.85	1.02	0.88
3	0.72	0.86	0.74	0.65	0.76	0.93	0.80
4	0.64	0.77	0.66	0.58	0.68	0.81	0.70

As shown in Exhibit 25-91, the demand-to-capacity ratio in the sixth section in the second analysis period is greater than one. As a result, queue formation and low space mean speeds are expected on this section. The demand-to-capacity ratios on the remaining segments are below one across all analysis periods.

Step 3: Delay Rate Estimation

In this step, demand-to-capacity ratios are used to determine delay rates for all sections of the facility across all analysis periods. FFS on the facility is 60 mi/h, and all demand-to-capacity ratios are below one. As a result, the delay rates for each section are found by using Equation 25-47.

$$\Delta_{RU_{i,t}} = \begin{cases} 0 & \frac{d_{i,t}}{c_i} < 0.72 \\ 121.35\left(\frac{d_{i,t}}{c_i}\right)^3 + (-184.84)\left(\frac{d_{i,t}}{c_i}\right)^2 + 83.21\left(\frac{d_{i,t}}{c_i}\right) + (-9.33) & 0.72 \le \frac{d_{i,t}}{c_i} \le 1 \end{cases}$$

For instance, the delay rate for Section 1 in the first analysis period is 0 s/mi, because its demand-to-capacity ratio of 0.717 is less than the 0.72 threshold used in Equation 25-47. Section 2's demand-to-capacity ratio is 0.86, which is greater than the threshold. Therefore, its delay rate is calculated as follows:

$$\Delta_{RU_{2,1}} = 121.35(0.86)^3 + (-184.84)(0.86)^2 + 83.21(0.86) + (0.86) = 2.8 \text{ s/mi}$$

Delay rates for other sections of the facility are determined in the same way and are summarized in Exhibit 25-92.

Exhibit 25-92
Example Problem 6: Delay Rates by Section and Analysis Period

Analysis Period	Delay Rate by Section (s/mi)						
	1	2	3	4	5	6	7
1	0.0	2.8	0.2	0.0	0.5	5.0	0.8
2	1.0	7.4	1.6	0.1	2.3	11.7	3.3
3	0.0	2.8	0.2	0.0	0.5	5.8	1.1
4	0.0	0.5	0.0	0.0	0.0	1.3	0.0

Step 4: Average Travel Time, Speed, and Density Calculations

Delay rates are used to compute travel times and, consequently, speeds. To determine a section's travel time, its travel rate is calculated by summing the section's travel rate under free-flow conditions and its delay rates for undersaturated and oversaturated conditions. This calculation is repeated for each section across all analysis periods. The following equations demonstrate the calculation for the first two sections during the first analysis period:

$$TR_{1,1} = \Delta_{RU_{1,1}} + \Delta_{RO_{1,1}} + TR_{FFS} = 0.00 + 0.00 + \frac{3{,}600}{S_{FFS}} = \frac{3{,}600}{60} = 60 \text{ s/mi}$$

$$TR_{2,1} = \Delta_{RU_{2,1}} + \Delta_{RO_{2,1}} + TR_{FFS} = 0.00 + 0.00 + \frac{3{,}600}{S_{FFS}} = 2.8 + \frac{3{,}600}{60}$$
$$= 62.8 \text{ s/mi}$$

Travel rates for all sections across all analysis periods are shown in Exhibit 25-93.

Exhibit 25-93
Example Problem 6:
Travel Rates by Section and Analysis Period

Analysis Period	Travel Rate by Section (s/mi)						
	1	2	3	4	5	6	7
1	60.0	62.8	60.2	60.0	60.5	65.0	60.8
2	61.0	67.4	61.6	60.1	62.3	71.7	63.3
3	60.0	62.8	60.2	60.0	60.5	65.8	61.1
4	60.0	60.5	60.0	60.0	60.0	61.3	60.0

Each section's travel time is calculated by multiplying its travel rate by its length. The results are presented in Exhibit 25-94.

Exhibit 25-94
Example Problem 6:
Average Travel Times by Section and Analysis Period

Analysis Period	Travel Time by Section (s)						
	1	2	3	4	5	6	7
1	60.0	62.8	60.2	30.0	60.5	32.5	60.8
2	61.0	67.4	61.6	30.0	62.3	35.8	63.3
3	60.0	62.8	60.2	30.0	60.5	32.9	61.1
4	60.0	60.5	60.0	30.0	60.0	30.7	60.0

Density is determined for each section across all analysis periods by dividing the section's demand by its speed (section length divided by travel time). The results are shown in Exhibit 25-95.

Exhibit 25-95
Example Problem 6:
Density by Section and Analysis Period

Analysis Period	Density by Section (pc/mi/ln)						
	1	2	3	4	5	6	7
1	27.5	31.1	28.5	23.3	29.5	34.2	30.6
2	31.1	37.2	32.4	25.9	33.8	41.2	35.4
3	27.5	31.1	28.5	23.3	29.5	35.2	31.3
4	24.4	26.7	25.2	20.7	26.0	28.7	26.8

Finally, the approach provides a high-level summary that includes a capacity assessment, the aggregated travel time, the space mean speed, the average facility density, the total queue length, and the facility LOS by analysis period, as shown in Exhibit 25-96.

Exhibit 25-96
Example Problem 6:
Facility Performance Summary

Analysis Period	High-Level Capacity Assessment	Travel Time (min)	Space Mean Speed (mi/h)	Average Facility Density (pc/mi/ln)	Total Queue Length (mi)	LOS
1	Undersaturated	6.1	58.9	29.2	0.0	D
2	Oversaturated	6.4	56.6	33.7	0.8	F
3	Undersaturated	6.1	58.8	29.4	0.0	D
4	Undersaturated	6.0	59.8	25.5	0.0	C

The average facility travel time in each time step is calculated by summing each section's travel time and dividing the result by 60 to convert the units to minutes. Space mean speed in each analysis period is then calculated by dividing the total facility length by the facility travel time in each analysis period. The facility density is a length-weighted average of each section's density, and the total queue length is the sum of each section's queue length. Finally, LOS is calculated based on the urban freeway density thresholds if the demand-to-capacity ratio is less than 1; otherwise, LOS is set to F if any section operates at a demand-to-capacity ratio greater than 1.0.

The facility is oversaturated during the second analysis period, with one of the sections experiencing a demand-to-capacity ratio greater than 1.0. The method estimates that a 0.8-mi queue will result from an active bottleneck. With at least one time interval operating at LOS F, it is recommended that a more detailed operational analysis of this facility be conducted to obtain a more accurate estimate of congestion patterns.

EXAMPLE PROBLEM 7: RELIABILITY EVALUATION OF AN EXISTING FREEWAY FACILITY

The Facility

This example problem uses the same 6-mi facility used in Example Problem 1. The facility consists of 11 segments with the properties indicated in Exhibit 25-97. Other facility characteristics are identical to those given in Example Problem 1, except that the study period in this example has been extended from 75 to 180 min. Exhibit 25-98 shows the facility geometry.

Exhibit 25-97
Example Problem 7: Freeway Facility

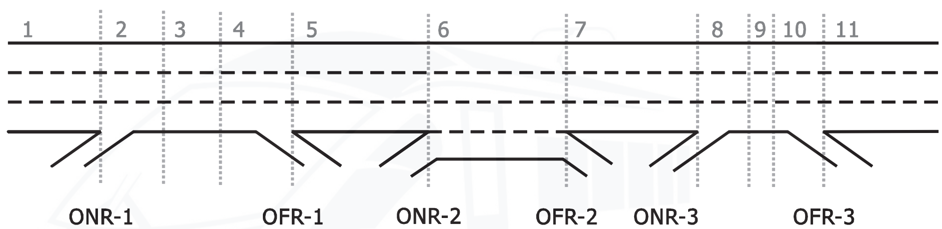


Exhibit 25-98
Example Problem 7: Geometry of Directional Freeway Facility

Segment No.	1	2	3	4	5	6	7	8	9	10	11
Segment type	B	ONR	B	OFR	B	B or W	B	ONR	R	OFR	B
Segment length (ft)	5,280	1,500	2,280	1,500	5,280	2,640	5,280	1,140	360	1,140	5,280
No. of lanes	3	3	3	3	3	4	3	3	3	3	3

Notes: B = basic freeway segment; W = weaving segment; ONR = on-ramp (merge) segment; OFR = off-ramp (diverge) segment; R = overlapping ramp segment.

Input Data

This example illustrates the use of defaults and lookup tables to substitute for desirable but difficult to obtain data. Minimum facility inputs for the example problem include the following.

Facility Geometry

All the geometric information about the facility normally required for an HCM freeway facility analysis (Chapters 10–14) is also required for a reliability analysis. These data are supplied as part of the base scenario.

Study Parameters

These parameters specify the study period, the reliability reporting period, and the date represented by the traffic demand data used in the base scenario.

The study period in this example is from 4 to 7 p.m., which covers the afternoon and early evening peak hour and shoulder periods. Recurring congestion is typically present in the study direction of this facility during that period, which is why it has been selected for reliability analysis. The reliability reporting period is set as all weekdays in the calendar year. (For simplicity of presentation in this example, holidays have not been removed from the reliability reporting period.) The demand data are reflective of AADT variations across the weekdays and months in a calendar year for the subject facility.

Base Demand

Demand flow rates in vehicles per hour are supplied for each 15-min analysis period in the base scenario. Care should be taken that demand data are measured upstream of any queued traffic. If necessary, demand can be estimated as the sum of departing volume and the change in the queue size at a recurring bottleneck.

Exhibit 25-99 provides the twelve 15-min demand flow rates required for the entire 3-h study period.

Exhibit 25-99
Example Problem 7: Demand Flow Rates (veh/h) by Analysis Period in the Base Data Set

Analysis Period	Demand Entry Flow Rate	ONR1	ONR2	ONR3	OFR1	OFR2	OFR3
1	3,095	270	270	270	180	270	180
2	3,595	360	360	360	270	360	270
3	4,175	360	450	450	270	360	270
4	4,505	450	540	450	270	360	270
5	4,955	540	720	540	360	360	270
6	5,225	630	810	630	270	360	450
7	4,685	360	360	450	270	360	270
8	3,785	180	270	270	270	180	180
9	3,305	180	270	270	270	180	180
10	2,805	180	270	270	270	180	180
11	2,455	180	180	180	270	180	180
12	2,405	180	180	180	180	180	180

Note: ONR = on-ramp; OFR = off-ramp.

Incident Data

Detailed incident logs are not available for this facility, but local data are available about the facility's crash rate: 150 crashes per 100 million VMT. An earlier study conducted by the state in which the facility is located found that an average of seven incidents occur for every crash.

Computational Steps

Base Data Set Analysis

The Chapter 10 freeway facilities core methodology is applied to the base data set to ensure the specified facility boundaries and study period are sufficient to cover any bottlenecks and queues. In addition, because incident data are supplied in the form of a facility crash rate, the VMT associated with the base data set are calculated so that incident probabilities can be calculated in a subsequent step. In this case, 71,501 vehicle miles of travel occur on the facility over the 3-h base study period. The performance measures normally output by the Chapter 10 methodology are compiled for each combination of segment and analysis period during the study period and stored for later use. Of particular note, the facility operates just under capacity, with a maximum demand-to-capacity ratio of 0.99 in Segments 7–10.

Incorporating Demand Variability

Exhibit 25-100 provides demand ratios relative to AADT by month and day derived from a permanent traffic recorder on the facility. The demand values for the seed file were collected on a Tuesday in November.

Exhibit 25-100
Example Problem 7: Demand Ratios Relative to AADT

Month	Monday	Tuesday	Wednesday	Thursday	Friday
January	0.822	0.822	0.839	0.864	0.965
February	0.849	0.849	0.866	0.892	0.996
March	0.921	0.921	0.939	0.967	1.080
April	0.976	0.976	0.995	1.025	1.145
May	0.974	0.974	0.993	1.023	1.142
June	1.022	1.022	1.043	1.074	1.199
July	1.133	1.133	1.156	1.191	1.329
August	1.033	1.033	1.054	1.085	1.212
September	1.063	1.063	1.085	1.117	1.248
October	0.995	0.995	1.016	1.046	1.168
November	0.995	0.995	1.016	1.046	1.168
December	0.979	0.979	0.998	1.028	1.148

Incorporating Weather Variability

In the absence of facility-specific weather data, the default weather data for the metropolitan area closest to the facility are used.

In the absence of local data, the default CAF and SAF for an FFS of 60 mi/h are used for each weather event. These values are applied in a later step to each scenario involving a weather event. Exhibit 25-101 summarizes the probabilities of each weather event by season, and Exhibit 25-102 summarizes the CAF, SAF, and event duration values associated with each weather event.

Exhibit 25-101
Example Problem 7: Weather Event Probabilities by Season

	Weather Event Probability by Season (%)			
Weather Event	Winter	Spring	Summer	Fall
Medium rain	0.80	1.01	0.71	0.86
Heavy rain	0.47	0.81	1.33	0.68
Light snow	0.91	0.00	0.00	0.00
Light–medium snow	0.29	0.00	0.00	0.00
Medium–heavy snow	0.04	0.00	0.00	0.00
Heavy snow	0.00	0.00	0.00	0.00
Severe cold	0.00	0.00	0.00	0.00
Low visibility	0.97	0.12	0.16	0.34
Very low visibility	0.00	0.00	0.00	0.00
Minimal visibility	0.44	0.10	0.00	0.03
Nonsevere weather	96.09	97.95	97.80	98.08

Note: Winter = December, January, and February; spring = March, April, and May; summer = June, July, and August; fall = September, October, and November.

Exhibit 25-102
Example Problem 7: CAF, SAF, and Event Duration Values Associated with Weather Events

Weather Event	CAF	SAF	Average Duration (min)
Medium rain	0.93	0.95	40.2
Heavy rain	0.86	0.93	33.7
Light snow	0.96	0.92	93.1
Light–medium snow	0.94	0.90	33.4
Medium–heavy snow	0.91	0.88	21.7
Heavy snow	0.78	0.86	7.3
Severe cold	0.92	0.95	0.0
Low visibility	0.90	0.95	76.2
Very low visibility	0.88	0.94	0.0
Minimal visibility	0.90	0.94	145
Nonsevere weather	1.00	1.00	N/A

Note: N/A = not applicable.

Incorporating Incident Variability

For an existing freeway facility such as this one, detailed incident logs would be desirable so that facility-specific monthly or seasonal probabilities of various incident severities could be determined. However, in this case, incident logs of sufficient detail are not available.

Therefore, incident probabilities and severities are estimated by the alternative method of using local crash rates and ratios of incidents to crashes, in combination with default values, by using Equation 25-77 through Equation 25-79. The expected number of incidents during a study period under a specified demand pattern is the product of the crash rate, the local incident-to-crash ratio, the demand volume during the study period, and the facility length. The crash rate is 150 crashes per 100 million VMT; the ratio of incidents to crashes is given as 7. The resulting incident frequencies for different months of the reliability reporting period are determined as shown in Exhibit 25-103.

Exhibit 25-103
Example Problem 7: Incident Frequencies by Month

Month	Incident Frequency
January	0.65
February	0.67
March	0.72
April	0.77
May	0.77
June	0.80
July	0.89
August	0.82
September	0.83
October	0.83
November	0.79
December	0.77

Results and Discussion

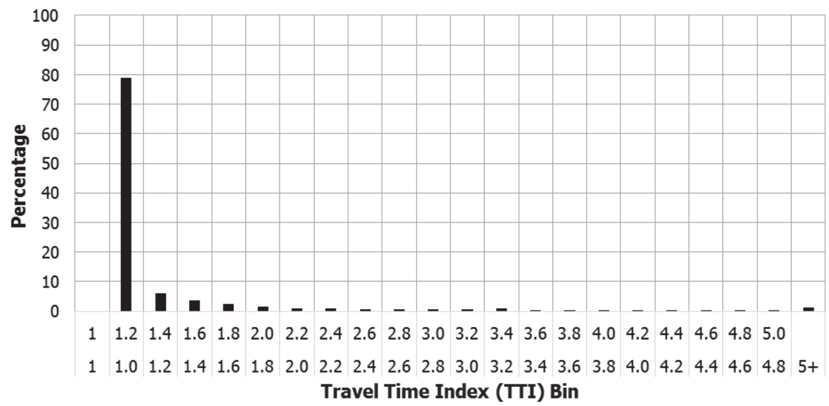
Exhibit 25-104 provides key reliability performance measure results for this example problem. The number of replications for each scenario was four, resulting in 240 scenarios. Exhibit 25-105 shows the generated probability and cumulative distributions of travel time index (TTI) for this example problem. A seed number of 1 was chosen to generate random numbers in the computational engine.

Exhibit 25-104
Example Problem 7: Summary Reliability Performance Measure Results

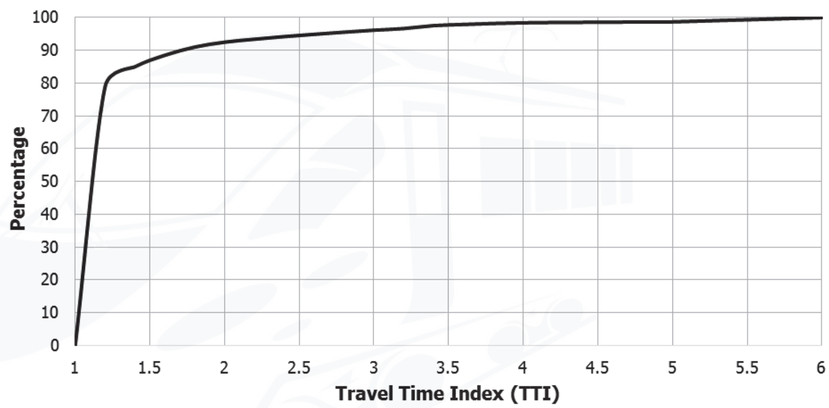
Reliability Performance Measure	Value from All Scenarios
TTI_{50}	1.03
TTI_{mean}	1.30
PTI (TTI_{95})	1.67
Maximum observed facility TTI (TTI_{max})	33.57
Misery index	5.76
Reliability rating	90.8%
Semi-standard deviation	2.05
Percentage VMT at TTI >2	2.95%

Note: PTI = planning time index; TTI = travel time index.

Exhibit 25-105
Example Problem 7:
VMT-Weighted TTI Probability and Cumulative Distribution Functions



(a) Probability Distribution Function



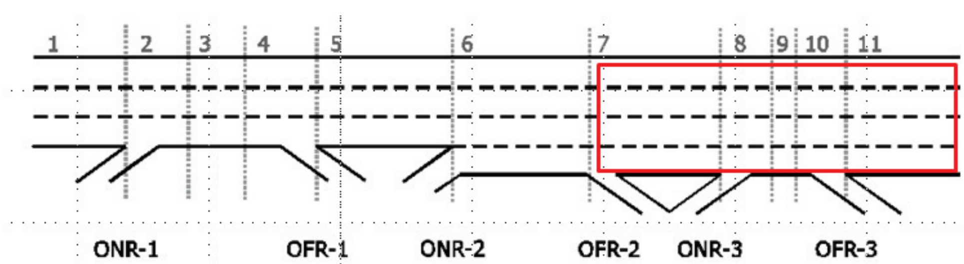
(b) Cumulative Distribution Function

EXAMPLE PROBLEM 8: RELIABILITY ANALYSIS WITH GEOMETRIC IMPROVEMENTS

The Facility

In this example, the freeway facility from Example Problem 6 is widened by a lane in Segments 7–11. These segments operated close to capacity in the base scenario and were definitely over capacity in scenarios with severe weather or incident conditions. The revised geometry also improves the operation of weaving Segment 6, because no lane changes are required of traffic entering at On-Ramp 2. Exhibit 25-106 provides a schematic of the freeway facility.

Exhibit 25-106
Example Problem 8:
Freeway Facility



Data Inputs

All the input data used in Example Problem 6 remain unchanged, except for the number of lanes on the facility. The extra lane creates the possibility of having a three-lane-closure incident scenario in the four-lane portion of the facility.

Results and Discussion

Exhibit 25-107 provides key reliability performance measure results for this example problem. The mean TTI across the reliability reporting period decreases from 1.54 to 1.18, corresponding to a speed improvement from 38.96 to 50.8 mi/h—more than a 10% increase and perhaps enough to justify the improvement, once non-reliability-related factors are taken into account. Similar results occur for most other performance measures.

Exhibit 25-107
Example Problem 8: Summary Reliability Performance Measure Results

Reliability Performance Measure	Value from All Scenarios
TTI_{50}	1.02
TTI_{mean}	1.18
PTI (TTI_{95})	1.17
Maximum observed facility TTI (TTI_{max})	33.5
Misery index	4.07
Reliability rating	97.56%
Semi-standard deviation	1.71
Percentage VMT at TTI >2	1.42%

Note: PTI = planning time index; TTI = travel time index.

EXAMPLE PROBLEM 9: EVALUATION OF INCIDENT MANAGEMENT

This example problem illustrates the analysis of a nonconstruction alternative that focuses on improved incident management strategies. In this example, the size of the motorist response fleet is increased and communication is improved between the various stakeholders (e.g., traffic management center, emergency responders, and motorist response fleet), allowing faster clearance of incidents than before.

Data Inputs

All the input data used in Example Problem 6 remain unchanged, except for the assumed incident durations and standard deviations. The default incident mean durations and standard deviations are reduced by 30% each for all incident severity types. Note that these values have been created for the purposes of this example problem and do not necessarily reflect results that would be obtained in an actual situation.

Results and Discussion

The key congestion and reliability statistics for this example problem are summarized in Exhibit 25-108. The mean TTI across the reliability reporting period decreases from 1.35 to 1.20, corresponding to a speed improvement from 44.4 to 50.0 mi/h—more than a 10% increase and perhaps enough to justify the improvement, once non-reliability-related factors are taken into account. Similar results occur for most other performance measures.

Exhibit 25-108
Example Problem 9:
Summary Reliability
Performance Measure Results

Reliability Performance Measure	Value from All Scenarios
TTI_{50}	1.03
TTI_{mean}	1.25
PTI (TTI_{95})	1.59
Maximum observed facility TTI (TTI_{max})	30.7
Misery index	4.88
Reliability rating	91.36%
Semi-standard deviation	1.77
Percentage VMT at TTI >2	2.4%

Note: PTI = planning time index; TTI = travel time index.

EXAMPLE PROBLEM 10: PLANNING-LEVEL RELIABILITY ANALYSIS

This example illustrates the planning-level reliability analysis methodology described in Chapter 11. The method estimates the mean and 95th percentile TTI, as well as the percentage of trips occurring below a speed of 45 mi/h.

The Facts

The segment under study has three lanes in the analysis direction, an FFS of 75 mi/h, and a peak hour speed of 62 mi/h. The volume-to-capacity ratio during the peak hour is 0.95.

Solution

The value of TTI_{mean} is calculated from Equation 11-1, and is a function of the recurring delay rate RDR and the incident delay rate IDR. These rates are calculated from Equation 11-2 and Equation 11-3, respectively.

$$RDR = \frac{1}{S} - \frac{1}{FFS}$$

$$RDR = \frac{1}{62} - \frac{1}{75} = 0.00280$$

$$IDR = [0.020 - (N-2) \times 0.003] \times X^{12}$$

$$IDR = [0.020 - (3-2) \times 0.003] \times (0.95)^{12} = 0.00919$$

TTI_{mean} can now be calculated as

$$TTI_{\text{mean}} = 1 + FFS \times (RDR + IDR)$$

$$TTI_{\text{mean}} = 1 + 75 \times (0.00280 + 0.00919)$$

$$TTI_{\text{mean}} = 1.899$$

TTI_{95} is calculated from Equation 11-4 as follows:

$$TTI_{95} = 1 + 3.67 \times \ln(TTI_{\text{mean}})$$

$$TTI_{95} = 1 + 3.67 \times \ln(1.899)$$

$$TTI_{95} = 3.353$$

Finally, the percentage of trips made at a speed below 45 mi/h is calculated with Equation 11-5.

$$PT_{45} = 1 - \exp\big(-1.5115 \times (TTI_{\text{mean}} - 1)\big)$$

$$PT_{45} = 1 - \exp\big(-1.5115 \times (1.899 - 1)\big)$$

$$PT_{45} = 74.3\%$$

EXAMPLE PROBLEM 11: ESTIMATING FREEWAY COMPOSITE GRADE OPERATIONS WITH THE MIXED-FLOW MODEL

This example problem addresses a composite grade section on a six-lane freeway. It illustrates how the mixed-flow model procedures can be applied to the case of composite grades.

The Facts

- Three segments with the following grades and lengths:
  - First segment: 1.5-mi basic segment on a 3% upgrade
  - Second segment: 2-mi basic segment on a 2% upgrade
  - Third segment: 1-mi basic segment on a 5% upgrade
- 5% SUTs and 10% TTs
- FFS of 65 mi/h
- 15-min mixed-traffic flow rate is 1,500 veh/h/ln (PHF = 1.0)

Comments

Chapter 26, Basic Freeway and Highway Segments: Supplemental, presents the procedure for estimating the speed on a single-grade basic freeway segment using the mixed-flow model. The task here is to estimate the speed by mode for each segment, along with the overall mixed-flow speed and travel time for the composite grade.

Step 1: Input Data

All input data are specified above.

Step 2: Capacity Assessment

The CAF for mixed flow allows for the conversion of auto-only capacities into mixed-traffic-stream capacities. It can be computed with Equation 25-53.

For the first segment,

$$CAF_{\text{mix},1} = CAF_{ao} - CAF_{T,\text{mix}} - CAF_{g,\text{mix},1}$$

There are four terms in the equation. The CAF for auto-only conditions CAF_{ao} is assumed to be 1, because no auto adjustments are necessary.

CAF for Truck Percentage

The truck effect term is computed from Equation 25-54.

$$CAF_{T,\text{mix}} = 0.53 \times {P_T}^{0.72} = 0.53 \times 0.15^{0.72} = 0.135$$

CAF for Grade Effect

The grade effect term is computed from Equation 25-55 and Equation 25-56. Given that the total truck percentage is 15%, the coefficient $\rho_{g,\text{mix}}$ is calculated as

$$\rho_{g,\text{mix}} = 0.126 - 0.03P_T = 0.126 - 0.03 \times 0.15 = 0.1215$$

and the CAF for grade effect for Segment 1 is calculated as

$$CAF_{g,\text{mix},1} = \rho_{g,\text{mix}} \times \max[0, 0.69 \times (e^{12.9g_j} - 1)] \times \max[0, 1.72 \times (1 - 1.71e^{-3.16d_j})]$$

$$CAF_{g,\text{mix},1} = 0.1215 \times \max[0, 0.69 \times (e^{12.9\times 0.03} - 1)] \times \max[0, 1.72 \times (1 - 1.71e^{-3.16\times 1.5})]$$

$$CAF_{g,\text{mix},1} = 0.067$$

Mixed-Flow CAF

The mixed-flow CAF for Segment 1 can now be calculated from Equation 25-53.

$$CAF_{\text{mix},j} = CAF_{ao} - CAF_{T,\text{mix}} - CAF_{g,\text{mix},1} = 1.000 - 0.135 - 0.067 = 0.798$$

Segment Capacity

The mixed-flow capacity of segment 1 is computed from the segment's auto-only capacity and mixed-flow CAF. The auto-only capacity is determined from an equation in Exhibit 12-6.

$$C_{ao} = 2{,}200 + 10(FFS - 50) = 2{,}200 + 10 \times (65 - 50) = 2{,}350 \text{ pc/h/ln}$$

Segment 1's mixed-flow capacity is then determined with Equation 25-57.

$$C_{\text{mix},1} = C_{ao} \times CAF_{\text{mix},1} = 2{,}350 \times 0.798 = 1{,}875 \text{ veh/h/ln}$$

Because the mixed-flow CAFs and capacities for Segments 2 and 3 can be computed by following the same procedure, the results are presented directly without showing the computational details.

$$CAF_{\text{mix},2} = CAF_{ao} - CAF_{T,\text{mix}} - CAF_{g,\text{mix},2} = 1 - 0.135 - 0.042 = 0.823$$

$$C_{\text{mix},2} = C_{ao} \times CAF_{\text{mix},2} = 2{,}350 \times 0.823 = 1{,}934 \text{ veh/h/ln}$$

$$CAF_{\text{mix},3} = CAF_{ao} - CAF_{T,\text{mix}} - CAF_{g,\text{mix},3} = 1 - 0.135 - 0.122 = 0.743$$

$$C_{\text{mix},3} = C_{ao} \times CAF_{\text{mix},3} = 2{,}350 \times 0.743 = 1{,}746 \text{ veh/h/ln}$$

As the mixed-flow demand of 1,500 veh/h/ln is less than the smallest of the three segment capacities, 1,746 veh/h/ln, the analysis can proceed.

Steps 3 to 6

Steps 3 through 6 are repeated for each segment, as shown below.

Segment 1

Step 3: Specify Initial Conditions

Because this is the first segment, an FFS of 65 mi/h is used as the initial truck kinematic spot travel time rate. The effect of traffic interactions on truck speed is accounted for in Step 4.

Step 4: Compute Truck Space-Based and Spot Travel Time Rates

Kinematic Spot Rates. The initial truck kinematic spot travel time rates for both SUTs and TTs are 65 mi/h. These rates are located on the curves representing a 3% upgrade starting from 75 mi/h (48 s/mi) in Exhibit 25-20 (SUTs) and Exhibit 25-21 (TTs).

The SUT and TT spot rates versus distance curves starting from 65 mi/h will be applied to obtain $\tau_{f,SUT,kin,1}$ and $\tau_{f,TT,kin,1}$. In Exhibit 25-20, 65 mi/h (55.4 s/mi) occurs about 4,100 ft into the 3% grade. After an SUT travels for 1.5 mi (7,920 ft) starting at an initial speed of 65 mi/h, its spot rate can be read at 12,020 ft. That distance is outside the plot range, but Exhibit 25-20 shows SUTs reach a crawl speed of 59 s/mi (61 mi/h) at around 10,000 ft. Therefore, the kinematic spot rate for SUTs at the end of the first segment $\tau_{f,SUT,kin,1}$ is 59 s/mi.

In Exhibit 25-21, 65 mi/h (55.4 s/mi) is found at about 2,100 ft. After a TT travels for 1.5 mi (7,920 ft) from an initial speed of 65 mi/h, its spot rate can be read at 12,020 ft, which is outside the plot range in Exhibit 25-21. However, similar to SUTs, TTs approach their crawl speed at 10,000 ft, namely 73 s/mi (49.3 mi/h).

Because this is the first segment, the initial truck kinematic rates $\tau_{i,SUT,kin,1}$ and $\tau_{i,TT,kin,1}$ are equivalent to the free-flow rate of 55.4 s/mi. Because $\tau_{i,SUT,kin,1}$ is less than $\tau_{f,SUT,kin,1}$ and $\tau_{i,TT,kin,1}$ is less than $\tau_{f,TT,kin,1}$, both types of trucks decelerate on Segment 1, from 65 to 61 mi/h for SUTs and from 65 to 49.3 mi/h for TTs.

Kinematic Space-Based Rates. Because this is the first segment, the space-based speed at 0 ft is the FFS of 65 mi/h. Therefore, the 65-mi/h curve is applied to obtain $\tau_{S,SUT,kin,1}$ and $\tau_{S,TT,kin,1}$.

The time for an SUT to travel 7,920 feet starting from 65 mi/h on a 3% grade can be read from Exhibit 25-A7 and is 87 s. The corresponding travel time for a TT can be read from Exhibit 25-A18 and is 99 s. The space mean rate at 7,920 ft for an SUT $\tau_{S,SUT,kin,65,7920}$ and a TT $\tau_{S,TT,kin,65,7920}$ starting from a FFS of 65 mi/h on a 3% grade can then be computed by Equation 25-58:

$$\tau_{S,SUT,kin,65,7920} = \frac{T_{SUT,65,7920}}{d_1} = \frac{87}{7{,}920/5{,}280} = 58 \text{ s/mi}$$

$$\tau_{S,TT,kin,65,7920} = \frac{T_{TT,65,7920}}{d_1} = \frac{99}{7{,}920/5{,}280} = 66 \text{ s/mi}$$

Auto-Only Speed for the Given Flow Rate. The auto-only space mean speed for the given flow rate is computed with Equation 25-63.

$$S_{ao} = \begin{cases} FFS & \dfrac{v_{\text{mix}}}{CAF_{\text{mix}}} \le BP_{ao} \\ FFS - \dfrac{\left(FFS - \dfrac{C_{ao}}{D_c}\right)\left(\dfrac{v_{\text{mix}}}{CAF_{\text{mix}}} - BP_{ao}\right)^2}{(C_{ao} - BP_{ao})^2} & \dfrac{v_{\text{mix}}}{CAF_{\text{mix}}} > BP_{ao} \end{cases}$$

The choice of equation depends on whether demand volumes are greater than or less than the breakpoint. An equation in Exhibit 12-6 is used to compute the breakpoint. For an auto-only condition, the CAF defaults to 1.0.

$$BP_{ao} = [1000 + 40 \times (75 - FFS)] \times CAF^2$$

$$BP_{ao} = [1000 + 40 \times (75 - 65)] \times 1^2 = 1{,}400 \text{ veh/h/ln}$$

As the demand volume of 1,500 veh/h/ln is greater than the breakpoint, the second of the two auto-only speed equations will be used. This equation requires knowing the auto-only capacity, which can be computed from Exhibit 12-6.

$$C_{ao} = 2{,}200 + 10 \times (65 - 50) = 2{,}350 \text{ pc/h/ln}$$

Then

$$S_{ao} = 65 - \frac{\left(65 - \frac{2{,}350}{45}\right)\left(\frac{1{,}500}{0.798} - 1{,}400\right)^2}{(2{,}350 - 1{,}400)^2} = 61.74 \text{ mi/h}$$

Traffic Interaction Term. The incremental traffic interaction term is computed with Equation 25-62.

$$\Delta\tau_{TI} = \left(\frac{3{,}600}{61.74} - \frac{3{,}600}{65}\right) \times \left(1 + 3\left(\frac{1}{0.798} - 1\right)\right) = 5.15 \text{ s/mi}$$

Actual Spot Rates. The actual spot travel time rates of SUTs and TTs at the end of Segment 1 are computed from Equation 25-60 and Equation 25-61, respectively.

$$\tau_{f,SUT,1} = \tau_{f,SUT,kin,1} + \Delta\tau_{TI} = 59 + 5.15 = 64.15 \text{ s/mi}$$

$$\tau_{f,TT,1} = \tau_{f,TT,kin,1} + \Delta\tau_{TI} = 73 + 5.15 = 78.15 \text{ s/mi}$$

The initial spot rates of SUTs and TTs in Segment 1 can also be computed from Equation 25-60 and Equation 25-61.

$$\tau_{i,SUT,1} = \tau_{i,SUT,kin,1} + \Delta\tau_{TI} = (3{,}600/65) + 5.15 = 60.5 \text{ s/mi}$$

$$\tau_{i,TT,1} = \tau_{i,TT,kin,1} + \Delta\tau_{TI} = (3{,}600/65) + 5.15 = 60.5 \text{ s/mi}$$

Actual Space-Based Rates. Equation 25-60 and Equation 25-61 are also used to calculate the actual space-based travel time rates for SUTs and TTs. The traffic interaction term is the same as the term used for the spot rate calculations.

$$\tau_{S,SUT,1} = \tau_{S,SUT,kin,1} + \Delta\tau_{TI} = 58 + 5.15 = 63.15 \text{ s/mi}$$

$$\tau_{S,TT,1} = \tau_{S,TT,kin,1} + \Delta\tau_{TI} = 66 + 5.15 = 71.15 \text{ s/mi}$$

Step 5: Compute Spot and Space-Based Travel Time Rates for Autos

Equation 25-64 is used to compute the spot-based travel time rate for automobiles on the basis of the kinematic truck spot rate at the end of the segment.

$$\begin{aligned}\tau_{f,a,1} = {} & \frac{3{,}600}{65} + 5.15 \\ & + \left[64.50 \times \left(\frac{1{,}500}{1{,}000}\right)^{0.77} \times 0.05^{0.34} \times \max\left(0, \frac{59}{100} - \frac{3{,}600}{65 \times 100}\right)^{1.53}\right] \\ & + \left[79.5 \times \left(\frac{1{,}500}{1{,}000}\right)^{0.81} \times 0.10^{0.56} \times \max\left(0, \frac{73}{100} - \frac{3{,}600}{65 \times 100}\right)^{1.32}\right]\end{aligned}$$

$$\tau_{f,a,1} = 63.8 \text{ s/mi}$$

When the initial auto spot travel time rate is computed, the trucks' kinematic spot rates are the same as the FFS, so the last two terms are 0. Therefore, Equation 25-64 can also be used to compute the initial auto spot rate, with the last two terms equal to 0.

$$\tau_{i,a,1} = \frac{3{,}600}{65} + 5.15 + 0 + 0$$

$$\tau_{i,a,1} = 60.5 \text{ s/mi}$$

It was determined in Step 4 that trucks decelerate in the first segment, so Equation 25-65 is used to compute the auto space-based rate on the basis of the kinematic truck space-based rates.

$$\tau_{S,a,1} = \frac{3{,}600}{65} + 5.15 + \left[100.42 \times \left(\frac{1{,}500}{1{,}000}\right)^{0.46} \times 0.05^{0.68} \times \max\left(0, \frac{58}{100} - \frac{3{,}600}{65 \times 100}\right)^{2.76}\right] + \left[110.64 \times \left(\frac{1{,}500}{1{,}000}\right)^{1.36} \times 0.10^{0.62} \times \max\left(0, \frac{66}{100} - \frac{3{,}600}{65 \times 100}\right)^{1.81}\right]$$

$$\tau_{S,a,1} = 61.3 \text{ s/mi}$$

Step 6: Compute Mixed-Flow Space-Based Travel Time Rate and Speed

The mixed-flow travel rate $\tau_{\text{mix},1}$ and the mixed speed $S_{\text{mix},1}$ are computed with Equation 25-67 and Equation 25-68, respectively.

$$\tau_{\text{mix},1} = 0.85 \times 61.3 + 0.05 \times 63.15 + 0.10 \times 71.15 = 62.4 \text{ s/mi}$$

$$S_{\text{mix},1} = \frac{3{,}600}{62.4} = 57.7 \text{ mi/h}$$

Segment 2

Step 3: Specify Initial Conditions

For the second segment, the initial truck kinematic spot travel time rates are the final truck kinematic spot rates from the preceding segment. These are 59 s/mi (61.0 mi/h) for SUTs and 73 s/mi (49.3 mi/h) for TTs.

Step 4: Compute Truck Space-Based and Spot Travel Time Rates

Kinematic Spot Rates. The initial truck kinematic spot travel time rates for both SUTs and TTs were determined in Step 3.

In Exhibit 25-20, the initial SUT kinematic spot rate of 59 s/mi (61.0 mi/h) occurs on the curve for a 2% upgrade, starting from 30 mi/h (120 s/mi) at approximately 4,000 ft along the curve. After an SUT travels for 2 mi (10,560 ft), its spot rate can be read at 14,560 ft, which is outside the plot range. However, Exhibit 25-20 shows SUTs approach their crawl speed of 67.9 mi/h (53 s/mi) on a 2% grade. Because the specified FFS is 65 mi/h, SUTs will maintain a speed of 65 mi/h (55.4 s/mi) when the kinematic spot speeds exceed 65 mi/h. Therefore, the SUT spot rate at the end of Segment 2, $\tau_{f,SUT,kin,2}$, is 55.4 s/mi.

In Exhibit 25-21, the initial TT kinematic spot rate of 73 s/mi (49.3 mi/h) occurs on the curve for a 2% upgrade, starting from 20 mi/h (180 s/mi) at approximately 3,360 ft. After a TT travels for 2 mi (10,560 ft), its spot rate can be read at 13,920 ft, which is outside the plot range. However, Exhibit 25-21 shows TTs reach their crawl speed of 57.1 mi/h (63 s/mi) on a 2% grade. Thus, the TT spot rate at the end of Segment 2, $\tau_{f,TT,kin,2}$, is 63 s/mi.

On this segment, the final SUT and TT kinematic rates are greater than the initial rates, so both truck types accelerate on the second grade. The nomographs for the time versus distance relationships are applicable to both cases where trucks are decelerating, and where they are accelerating. Acceleration is evident if the time required to cover a given distance is reducing as the distance increases.

Kinematic Space-Based Rates. The kinematic space-based speeds at 0 ft into Segment 2 equal the final kinematic spot speeds of Segment 1.

For SUTs, the final kinematic spot speed of Segment 1 was 61.0 mi/h (59 s/mi). As this speed is within 2.5 mi/h of 60 mi/h, Exhibit 25-A6 is used to obtain the SUT kinematic space-based travel time rate $\tau_{S,SUT,kin,2}$. The time for an SUT to travel 10,000 ft starting from an FFS of 60 mi/h on a 2% grade can be read from Exhibit 25-A6 and is 105 s.

For TTs, the final kinematic spot speed of Segment 1 was 49.3 mi/h (73 s/mi). As this speed is within 2.5 mi/h of 50 mi/h, Exhibit 25-A15 is applied to obtain the TT kinematic space-based rate $\tau_{S,TT,kin,2}$. The time for a TT to travel 10,000 ft starting from an FFS of 50 mi/h on a 2% grade can be read from Exhibit 25-A15 and is 125 s.

The space mean travel time rates for SUTs and TTs can now be computed by Equation 25-58.

$$\tau_{S,SUT,kin,60,10000} = \frac{T_{SUT,60,10000}}{d_2} = \frac{105}{10{,}000/5{,}280} = 55.4 \text{ s/mi}$$

$$\tau_{S,TT,kin,50,10000} = \frac{T_{TT,50,10000}}{d_2} = \frac{125}{10{,}000/5{,}280} = 66.0 \text{ s/mi}$$

The SUT and TT kinematic rates at a distance of 2 mi (10,560 ft) can be computed from Equation 25-59. The δ values for SUTs (0.0104) and TTs (0.0136) can be read from Exhibit 25-24 and Exhibit 25-25, respectively. The rates are computed as follows:

$$\tau_{S,SUT,kin,60,10560} = \frac{105}{2} + 0.0105 \times \left(1 - \frac{10{,}000}{2 \times 5{,}280}\right) \times 5{,}280 = 55.4 \text{ s/mi}$$

$$\tau_{S,TT,kin,60,10560} = \frac{125}{2} + 0.0118 \times \left(1 - \frac{10{,}000}{2 \times 5{,}280}\right) \times 5{,}280 = 65.8 \text{ s/mi}$$

Auto-Only Speed for the Given Flow Rate. The auto-only space mean speed for the given flow rate is computed with Equation 25-63. The breakpoint of the speed–flow curve was already determined to be 1,400 veh/h/ln, as part of the computations for the first segment. Thus,

$$S_{ao} = 65 - \frac{\left(65 - \frac{2{,}350}{45}\right)\left(\frac{1{,}500}{0.823} - 1{,}400\right)^2}{(2{,}350 - 1{,}400)^2} = 62.46 \text{ mi/h}$$

Traffic Interaction Term. The incremental traffic interaction term is computed by Equation 25-62.

$$\Delta\tau_{TI} = \left(\frac{3{,}600}{62.46} - \frac{3{,}600}{65}\right) \times \left(1 + 3\left(\frac{1}{0.823} - 1\right)\right) = 3.71 \text{ s/mi}$$

Actual Spot Rates. The actual spot rates of SUTs and TTs at the end of Segment 2 are computed from Equation 25-60 and Equation 25-61, respectively.

$$\tau_{f,SUT,2} = \tau_{f,SUT,kin,2} + \Delta\tau_{TI} = 55.4 + 3.71 = 59.11 \text{ s/mi}$$

$$\tau_{f,TT,2} = \tau_{f,TT,kin,2} + \Delta\tau_{TI} = 63 + 3.71 = 66.71 \text{ s/mi}$$

Similarly, the space-based rates are

$$\tau_{S,SUT,2} = \tau_{S,SUT,kin,2} + \Delta\tau_{TI} = 55.4 + 3.71 = 59.11 \text{ s/mi}$$
$$\tau_{S,TT,2} = \tau_{S,TT,kin,2} + \Delta\tau_{TI} = 65.8 + 3.71 = 69.51 \text{ s/mi}$$

Step 5: Compute Spot and Space-Based Travel Time Rates for Autos

Equation 25-64 is used to compute the spot-based travel time rate for automobiles.

$$\tau_{f,a,2} = \frac{3{,}600}{65} + 3.71 + \left[64.50 \times \left(\frac{1{,}500}{1{,}000}\right)^{0.77} \times 0.05^{0.34} \times \max\left(0, \frac{55.4}{100} - \frac{3{,}600}{65 \times 100}\right)^{1.53}\right] + \left[79.5 \times \left(\frac{1{,}500}{1{,}000}\right)^{0.81} \times 0.10^{0.56} \times \max\left(0, \frac{66.0}{100} - \frac{3{,}600}{65 \times 100}\right)^{1.32}\right]$$

$$\tau_{f,a,2} = 60.1 \text{ s/mi}$$

In this case, the auto spot rate of 60.1 s/mi is higher than the SUT spot rate of 59.1 s/mi. As the auto spot rate should always be less than or equal to the truck spot rate, the auto spot rate is set equal to 59.11 s/mi.

In Step 4, it was determined that trucks accelerate in Segment 2, so Equation 25-66 is used to compute the auto space-based rate.

$$\tau_{S,a,2} = \frac{3{,}600}{65} + 3.71 + \left[54.72 \times \left(\frac{1{,}500}{1{,}000}\right)^{1.16} \times 0.05^{0.28} \times \max\left(0, \frac{55.4}{100} - \frac{3{,}600}{65 \times 100}\right)^{1.73}\right] + \left[69.72 \times \left(\frac{1{,}500}{1{,}000}\right)^{1.32} \times 0.10^{0.61} \times \max\left(0, \frac{65.8}{100} - \frac{3{,}600}{65 \times 100}\right)^{1.33}\right]$$

$$\tau_{S,a,2} = 60.5 \text{ s/mi}$$

Step 6: Compute Mixed-Flow Space-Based Travel Time Rate and Speed

The mixed-flow travel rate $\tau_{\text{mix},2}$ and the mixed speed $S_{\text{mix},2}$ are computed with Equation 25-67 and Equation 25-68, respectively.

$$\tau_{\text{mix},2} = 0.85 \times 61.4 + 0.05 \times 62.01 + 0.10 \times 73.51 = 62.6 \text{ s/mi}$$

$$S_{\text{mix},2} = \frac{3{,}600}{61.3} = 58.7 \text{ mi/h}$$

Segment 3

Step 3: Specify Initial Conditions

The initial truck kinematic spot travel time rates for Segment 3 are the final truck kinematic spot rates for Segment 2. These are 55.4 s/mi (65 mi/h) for SUTs and 63.0 s/mi (57.1 mi/h) for TTs.

Step 4: Compute Truck Space-Based and Spot Travel Time Rates

Kinematic Spot Rates. The initial truck kinematic spot travel time rates for both SUTs and TTs were determined in Step 3.

In Exhibit 25-20, the initial SUT kinematic spot rate of 55.4 s/mi (65 mi/h) occurs on the curve for a 5% upgrade, starting from 75 mi/h (48 s/mi) at approximately 1,500 ft along the curve. After an SUT travels 1 mi (5,280 ft), its spot rate can be read at 6,780 ft and is approximately 75 s/mi (48 mi/h). Thus, the SUT spot rate at the end of Segment 3 is 75 s/mi.

In Exhibit 25-21, the initial TT kinematic spot rate of 63 s/mi (57.1 mi/h) occurs on the curve for a 5% upgrade, starting from 75 mi/h (48 s/mi) at approximately 2,050 ft along the curve. After a TT travels 1 mi (5,280 ft), its spot rate can be read at 7,330 ft and is approximately 103 s/mi (35.0 mi/h). Thus, the TT spot rate at the end of Segment 3 is 103 s/mi.

In Segment 3, the initial kinematic rates for both truck types are less than the final kinematic rates. Therefore, both truck types decelerate in Segment 3.

Kinematic Space-Based Rates. The kinematic space-based speeds at 0 ft into Segment 3 equal the final kinematic spot speeds of Segment 2.

The final kinematic spot speed of SUTs in Segment 2 was 65 mi/h (55.4 s/mi). Exhibit 25-A7 is therefore used to obtain the SUT kinematic space-based rate $\tau_{S,SUT,kin,3}$. The travel time for SUTs at 5,280 ft, starting from 65 mi/h on a 5% grade, can be read from Exhibit 25-A7 and equals 67 s.

The final kinematic spot speed of TTs in Segment 2 was 57.2 mi/h (63.0 s/mi). As this value is within 2.5 mi/h of 55 mi/h, Exhibit 25-A16 is applied to obtain the TT kinematic space-based rate $\tau_{S,TT,kin,3}$. The travel time for TTs at 5,280 ft, starting from an FFS of 55 mi/h on a 5% grade, can be read from Exhibit 25-A16 and equals 89 s.

The space mean rate at 5,280 ft for SUTs and TTs can be computed by Equation 25-58.

$$\tau_{S,SUT,kin,65,5280} = \frac{T_{SUT,65,5280}}{d_3} = \frac{67}{5{,}280/5{,}280} = 67.0 \text{ s/mi}$$

$$\tau_{S,TT,kin,55,5280} = \frac{T_{TT,55,5280}}{d_3} = \frac{89}{5{,}280/5{,}280} = 89.0 \text{ s/mi}$$

Auto-Only Speed for the Given Flow Rate. The auto-only space mean speed for the given flow rate is computed with Equation 25-63. The breakpoint of the speed–flow curve was already determined to be 1,400 veh/h/ln as part of the computations for the first segment. Thus

$$S_{ao} = 65 - \frac{\left(65 - \frac{2{,}350}{45}\right)\left(\frac{1{,}500}{0.743} - 1{,}400\right)^2}{(2{,}350 - 1{,}400)^2} = 59.58 \text{ mi/h}$$

Traffic Interaction Term. The incremental traffic interaction term is computed by Equation 25-62.

$$\Delta\tau_{TI} = \left(\frac{3{,}600}{59.58} - \frac{3{,}600}{65}\right) \times \left(1 + 3\left(\frac{1}{0.743} - 1\right)\right) = 10.27 \text{ s/mi}$$

Actual Spot Rates. The actual spot rates of SUTs and TTs at the end of Segment 2 are computed from Equation 25-60 and Equation 25-61, respectively.

$$\tau_{f,SUT,3} = \tau_{f,SUT,kin,3} + \Delta\tau_{TI} = 75 + 10.27 = 85.27 \text{ s/mi}$$

$$\tau_{f,TT,3} = \tau_{f,TT,kin,3} + \Delta\tau_{TI} = 103 + 10.27 = 113.27 \text{ s/mi}$$

Similarly the space-based rates are:

$$\tau_{S,SUT,3} = \tau_{S,SUT,kin,3} + \Delta\tau_{TI} = 67.0 + 10.27 = 77.27 \text{ s/mi}$$

$$\tau_{S,TT,3} = \tau_{S,TT,kin,3} + \Delta\tau_{TI} = 89.0 + 10.27 = 99.27 \text{ s/mi}$$

Step 5: Compute Spot and Space-Based Travel Time Rates for Autos

Equation 25-64 is used to compute the spot-based travel time rate for automobiles.

$$\begin{aligned}\tau_{f,a,3} = {} & \frac{3{,}600}{65} + 10.27 \\ & + \left[64.50 \times \left(\frac{1{,}500}{1{,}000}\right)^{0.77} \times 0.05^{0.34} \times \max\left(0, \frac{75}{100} - \frac{3{,}600}{65 \times 100}\right)^{1.53}\right] \\ & + \left[79.5 \times \left(\frac{1{,}500}{1{,}000}\right)^{0.81} \times 0.10^{0.56} \times \max\left(0, \frac{103}{100} - \frac{3{,}600}{65 \times 100}\right)^{1.32}\right]\end{aligned}$$

$$\tau_{f,a,3} = 79.7 \text{ s/mi}$$

In Step 4, it was determined that trucks decelerate in Segment 3, so Equation 25-65 is used to compute the auto space-based rate.

$$\begin{aligned}\tau_{S,a,3} = {} & \frac{3{,}600}{65} + 10.27 \\ & + \left[100.42 \times \left(\frac{1{,}500}{1{,}000}\right)^{0.46} \times 0.05^{0.68} \times \max\left(0, \frac{67.0}{100} - \frac{3{,}600}{65 \times 100}\right)^{2.76}\right] \\ & + \left[110.64 \times \left(\frac{1{,}500}{1{,}000}\right)^{1.36} \times 0.10^{0.62} \times \max\left(0, \frac{89.0}{100} - \frac{3{,}600}{65 \times 100}\right)^{1.81}\right]\end{aligned}$$

$$\tau_{S,a,3} = 72.1 \text{ s/mi}$$

Step 6: Compute Mixed-Flow Space-Based Travel Time Rate and Speed

The mixed-flow travel rate $\tau_{\text{mix},3}$ and the mixed speed $S_{\text{mix},3}$ are computed using Equation 25-67 and Equation 25-68, respectively.

$$\tau_{\text{mix},3} = 0.85 \times 72.1 + 0.05 \times 77.27 + 0.10 \times 99.27 = 75.1 \text{ s/mi}$$

$$S_{\text{mix},3} = \frac{3{,}600}{75.1} = 47.9 \text{ mi/h}$$

Step 7: Overall Results

Now that results have been developed for all three segments, the overall performance of the composite grade can be computed. The mixed-flow travel time for each segment is computed with Equation 25-69.

$$t_{\text{mix},1} = \frac{3{,}600 d_1}{S_{\text{mix},1}} = \frac{3{,}600 \times 1.5}{57.7} = 93.6 \text{ s}$$

$$t_{\text{mix},2} = \frac{3{,}600 d_2}{S_{\text{mix},2}} = \frac{3{,}600 \times 2}{58.7} = 122.7 \text{ s}$$

$$t_{\text{mix},3} = \frac{3{,}600 d_3}{S_{\text{mix},3}} = \frac{3{,}600 \times 1}{47.9} = 75.2 \text{ s}$$

The overall mixed-flow travel time $t_{\text{mix},oa}$ is the sum of the mixed-flow travel times for all three segments and equals 294 s. Equation 25-70 can be used to compute the mixed-flow speed.

$$S_{\text{mix},oa} = \frac{3{,}600 d_{oa}}{t_{\text{mix},oa}} = \frac{3600 \times 4.5}{291.5} = 55.6 \text{ mi/h}$$

Exhibit 25-109 shows the spot speeds of all the segments in the example.

Exhibit 25-109
Example Problem 11:
Spot Speeds of All Segments

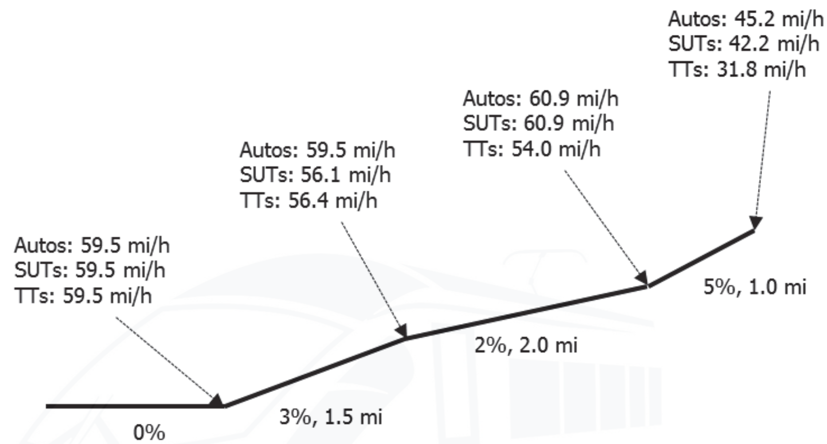


Exhibit 25-110 shows the space mean speeds of all the segments in the example.

Exhibit 25-110
Example Problem 11: Space Mean Speeds of All Segments

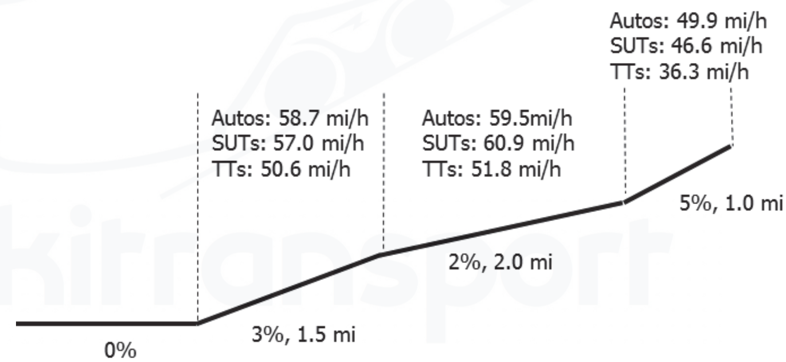
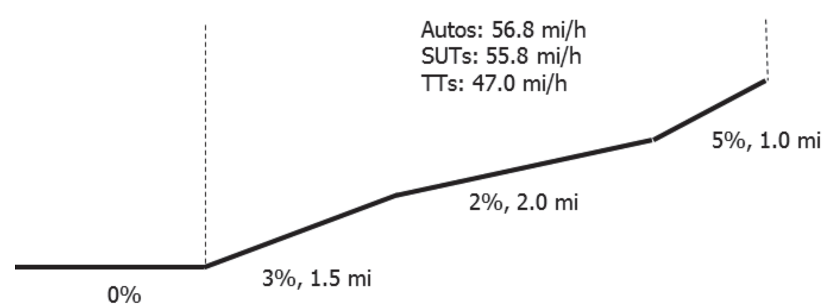


Exhibit 25-111 shows the overall space mean speeds of all the segments in the example.

Exhibit 25-111
Example Problem 11: Overall Space Mean Speeds of All Segments



12. REFERENCES

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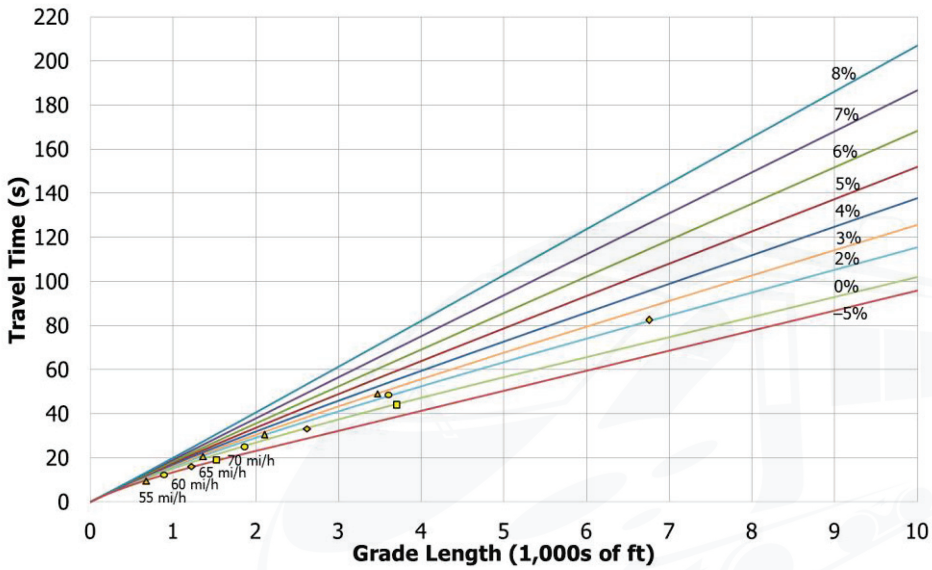
APPENDIX A: TRUCK PERFORMANCE CURVES

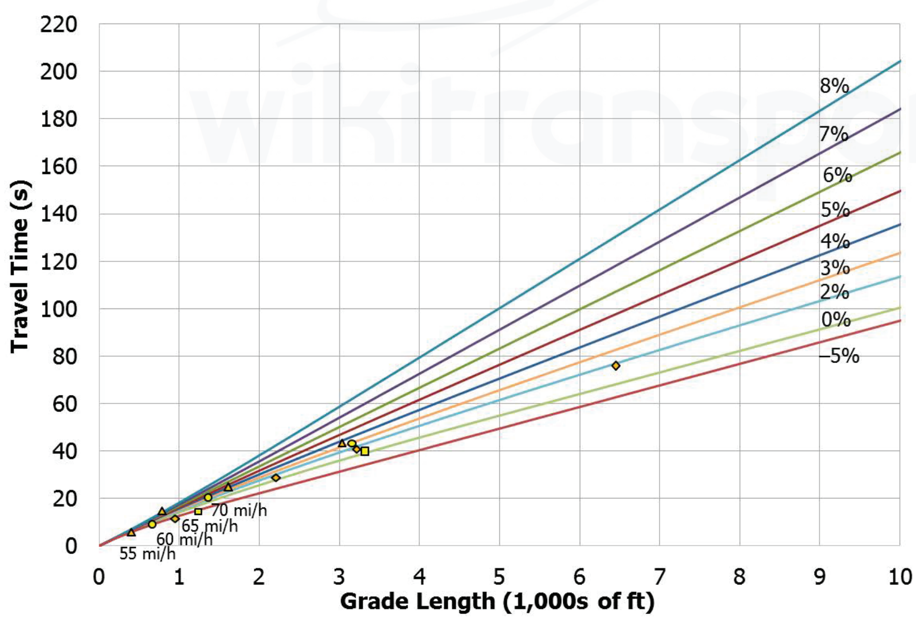
This appendix provides travel time versus distance curves for SUTs for initial speeds between 35 and 75 mi/h in 5-mi/h increments. Curves for SUTs for 30- and 70-mi/h initial speeds are presented in Section 7 as Exhibit 25-23 and Exhibit 25-22, respectively. The appendix also provides travel time versus distance curves for TTs for initial speeds between 20 and 75 mi/h in 5-mi/h increments.

Exhibit 25-A1
SUT Travel Time Versus Distance Curves for 35-mi/h Initial Speed



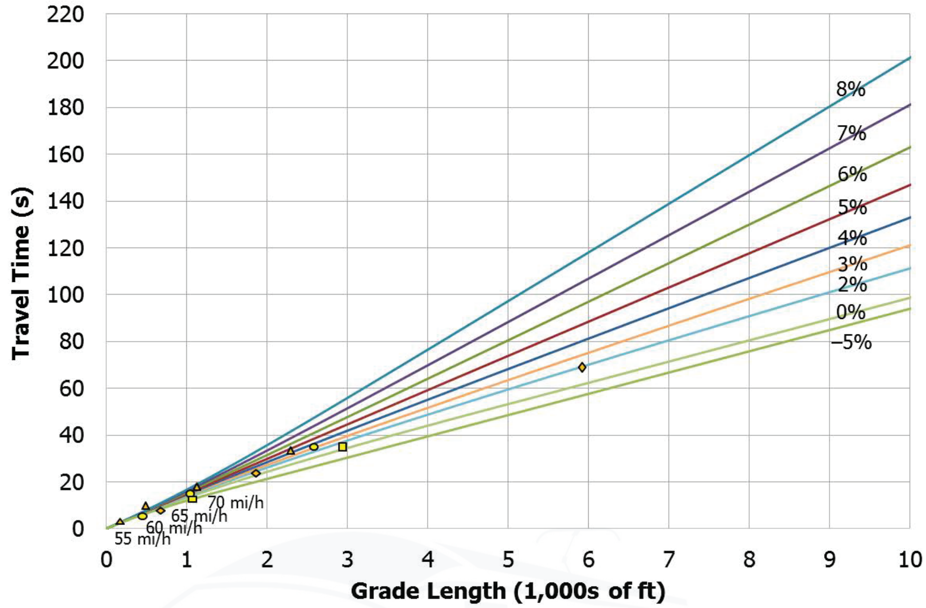
Notes: Curves in this graph assume a weight-to-horsepower ratio of 100.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 25-A2
SUT Travel Time Versus Distance Curves for 40-mi/h Initial Speed



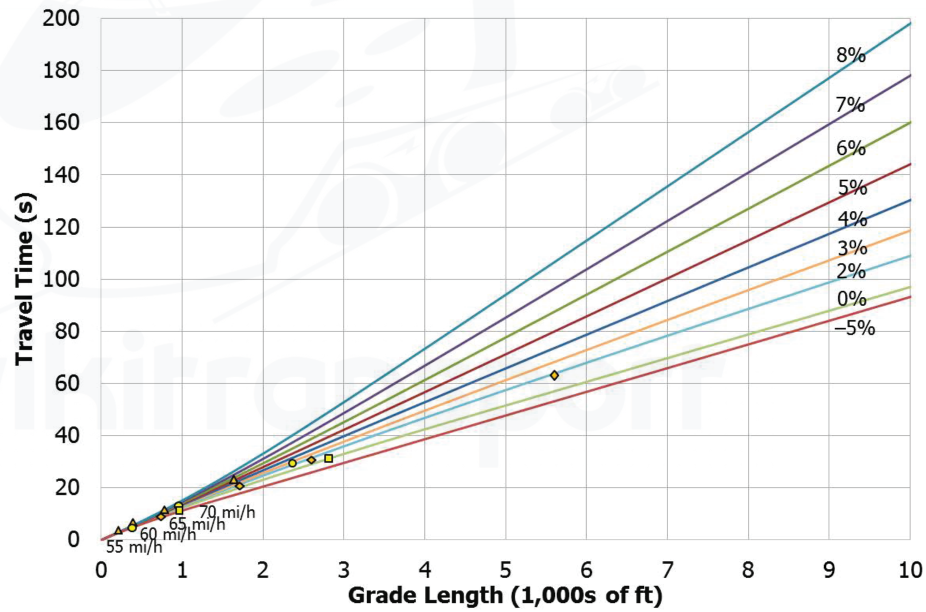
Notes: Curves in this graph assume a weight-to-horsepower ratio of 100.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 25-A3
SUT Travel Time Versus Distance Curves for 45-mi/h Initial Speed



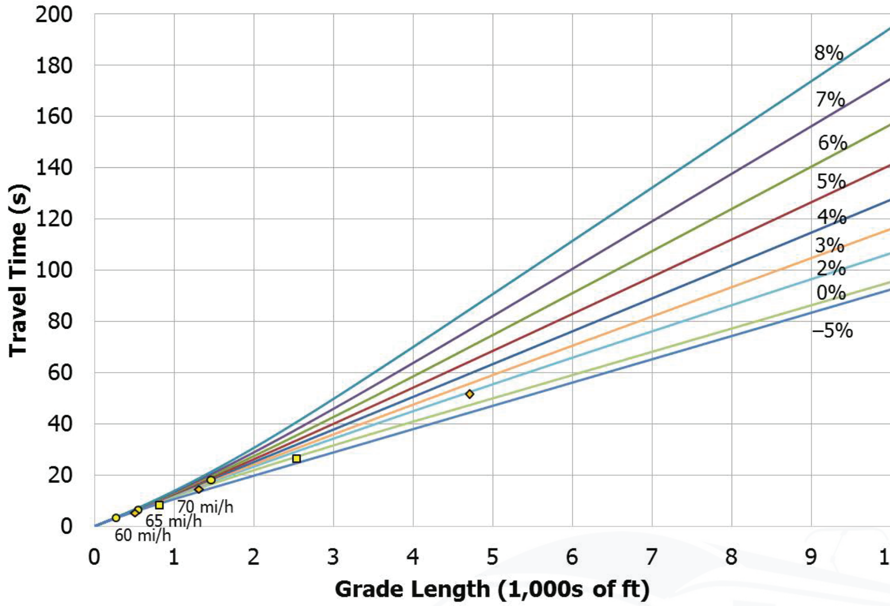
Notes: Curves in this graph assume a weight-to-horsepower ratio of 100.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 25-A4
SUT Travel Time Versus Distance Curves for 50-mi/h Initial Speed



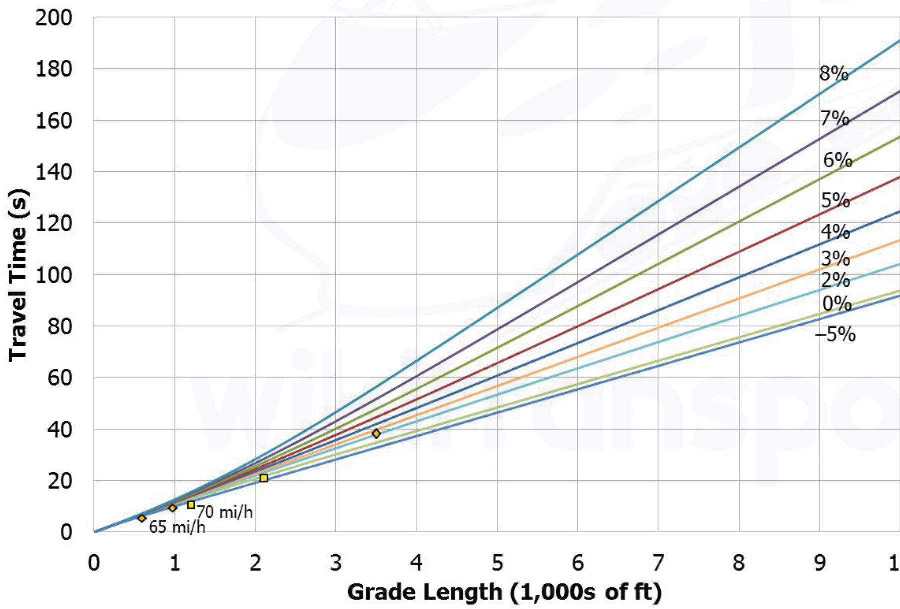
Notes: Curves in this graph assume a weight-to-horsepower ratio of 100.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 25-A5
SUT Travel Time Versus Distance Curves for 55-mi/h Initial Speed



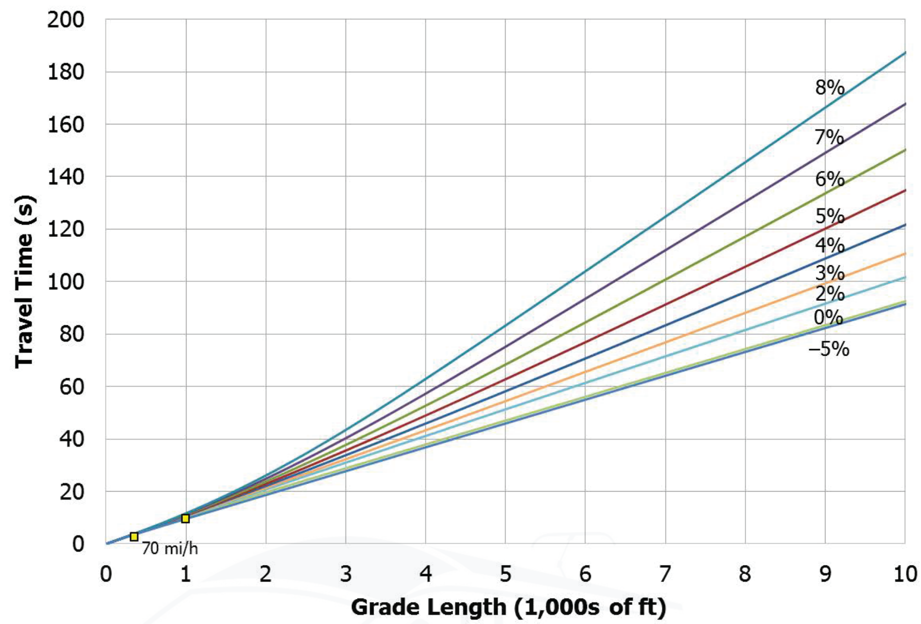
Notes: Curves in this graph assume a weight-to-horsepower ratio of 100.
Circles indicate where a truck reaches 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 25-A6
SUT Travel Time Versus Distance Curves for 60-mi/h Initial Speed



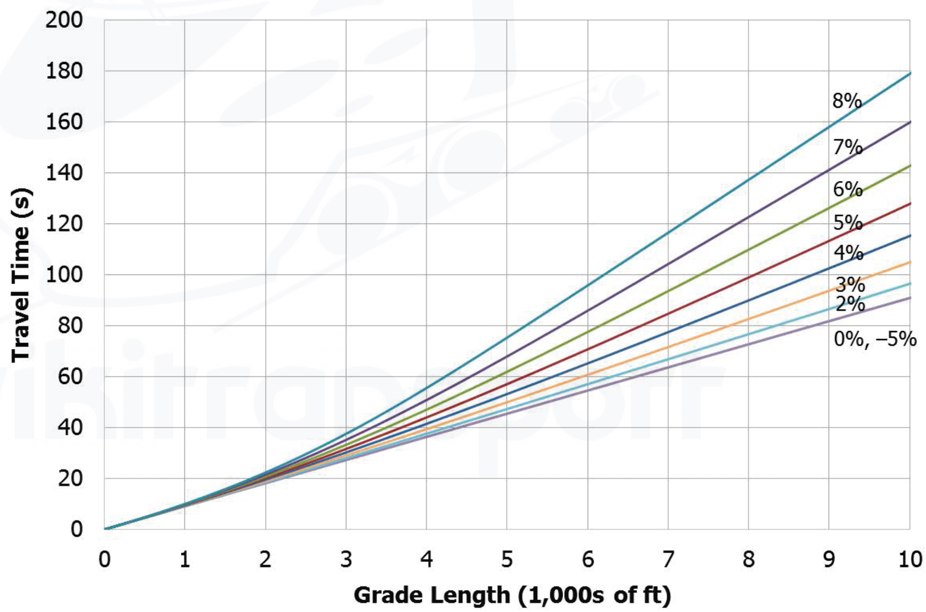
Notes: Curves in this graph assume a weight-to-horsepower ratio of 100.
Diamonds indicate where a truck reaches 65 mi/h and squares indicate 70 mi/h.

Exhibit 25-A7
SUT Travel Time Versus Distance Curves for 65-mi/h Initial Speed



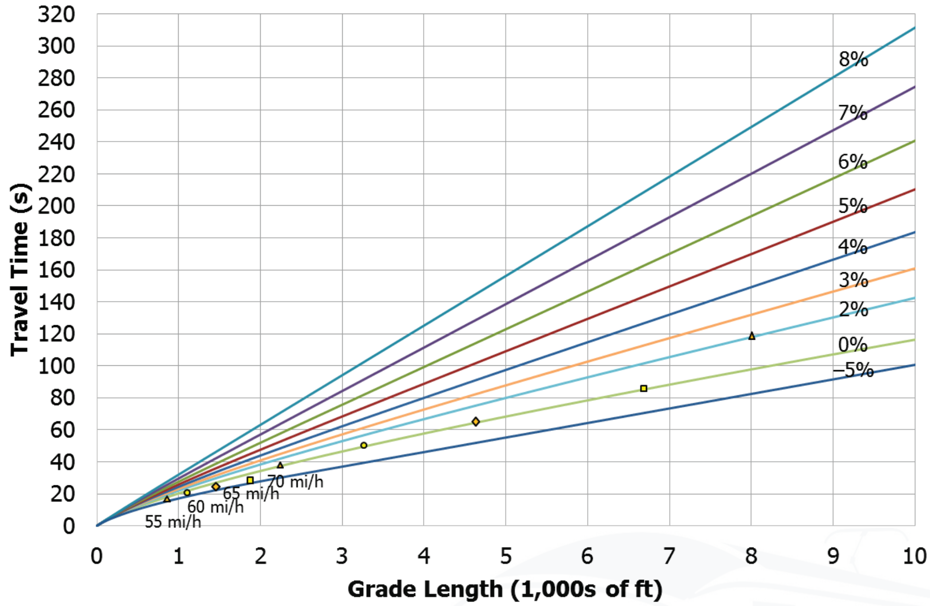
Notes: Curves in this graph assume a weight-to-horsepower ratio of 100.
Squares indicate where a truck reaches 70 mi/h.

Exhibit 25-A8
SUT Travel Time Versus Distance Curves for 75-mi/h Initial Speed



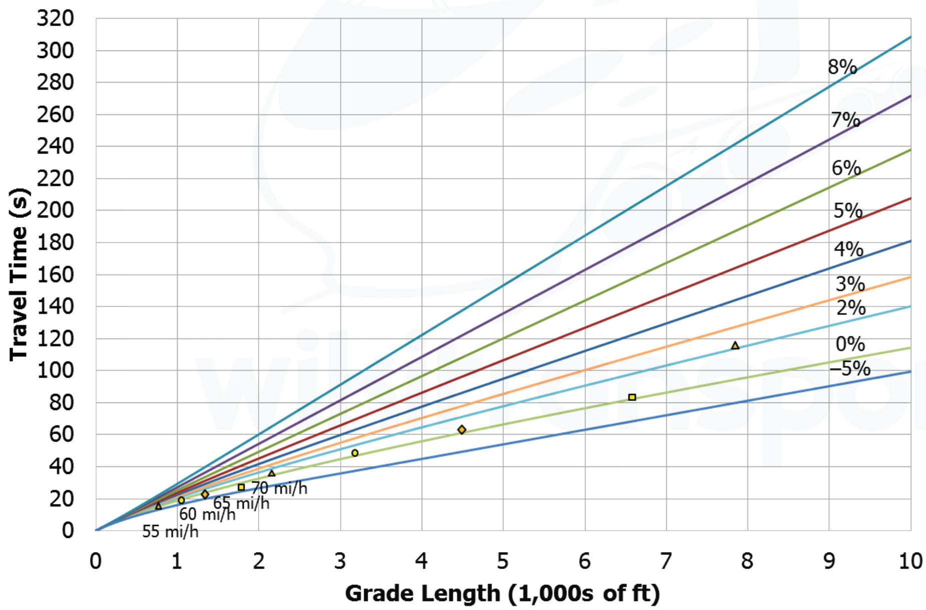
Note: Curves in this graph assume a weight-to-horsepower ratio of 100.

Exhibit 25-A9
TT Travel Time Versus Distance Curves for 20-mi/h Initial Speed



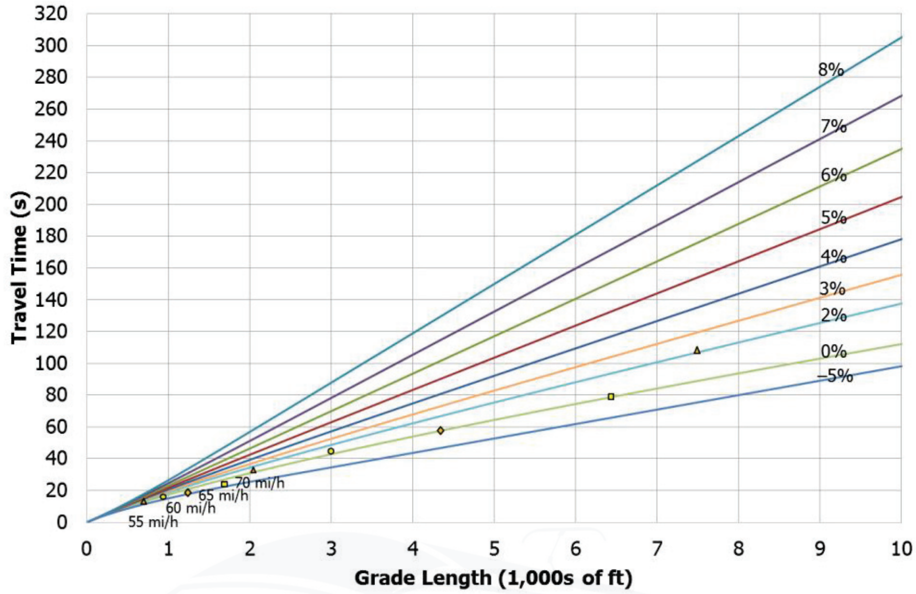
Notes: Curves in this graph assume a weight-to-horsepower ratio of 150.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 25-A10
TT Travel Time Versus Distance Curves for 25-mi/h Initial Speed



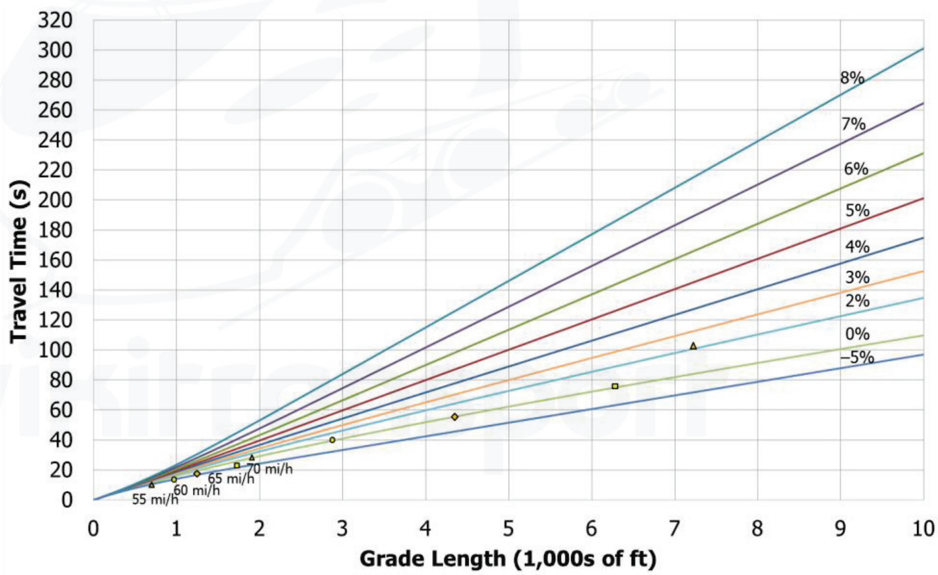
Notes: Curves in this graph assume a weight-to-horsepower ratio of 150.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 25-A11
TT Travel Time Versus Distance Curves for 30-mi/h Initial Speed



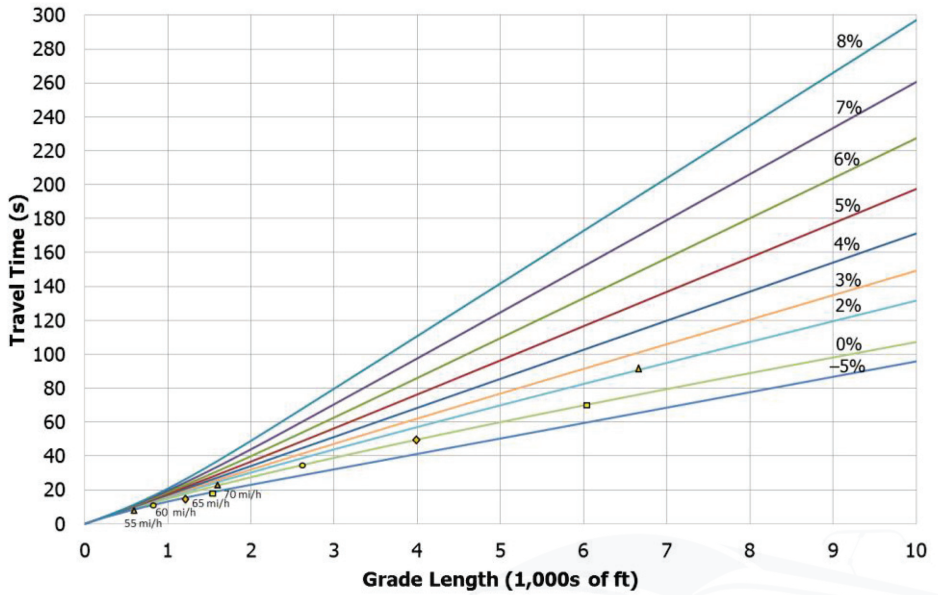
Notes: Curves in this graph assume a weight-to-horsepower ratio of 150.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 25-A12
TT Travel Time Versus Distance Curves for 35-mi/h Initial Speed



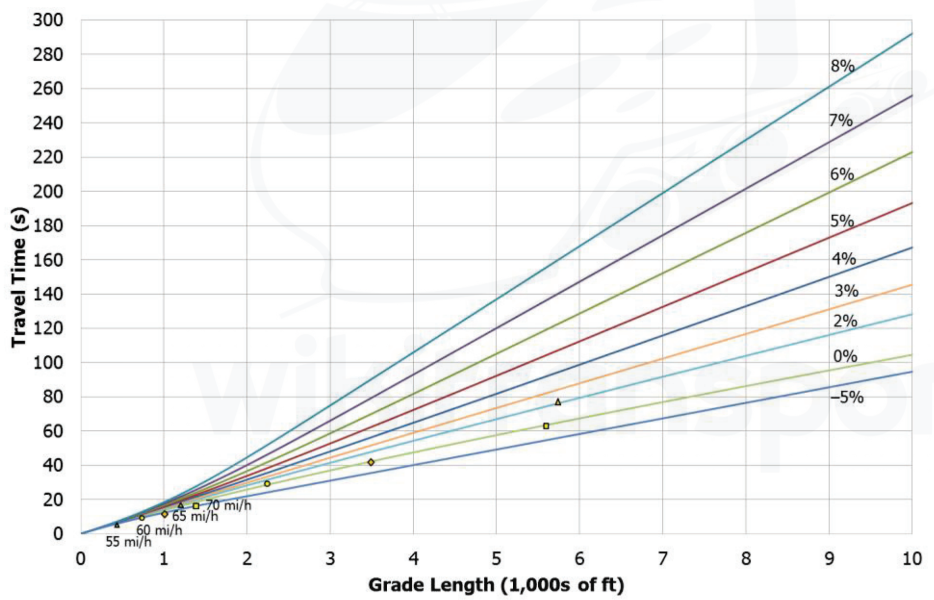
Notes: Curves in this graph assume a weight-to-horsepower ratio of 150.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 25-A13
TT Travel Time Versus Distance Curves for 40-mi/h Initial Speed



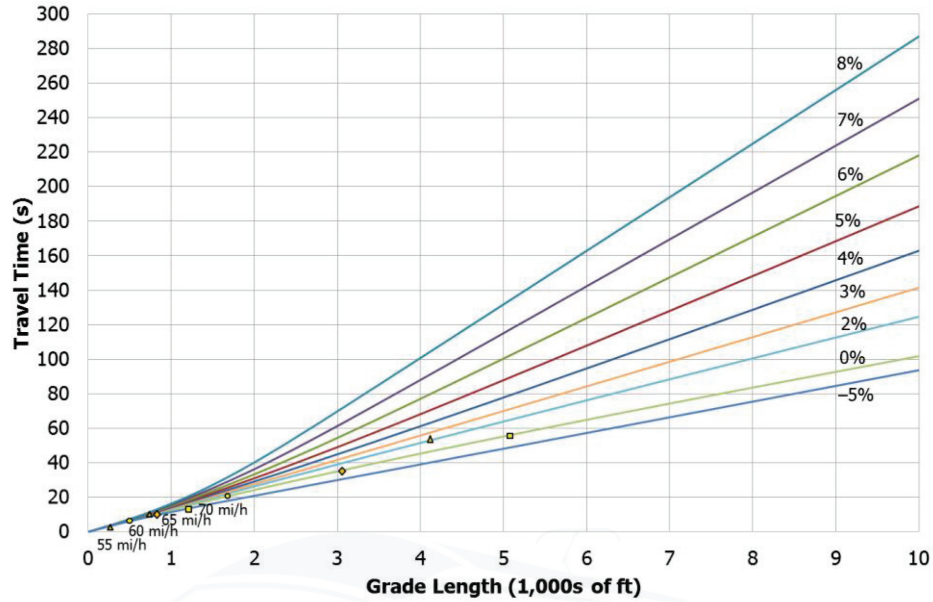
Notes: Curves in this graph assume a weight-to-horsepower ratio of 150.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 25-A14
TT Travel Time Versus Distance Curves for 45-mi/h Initial Speed



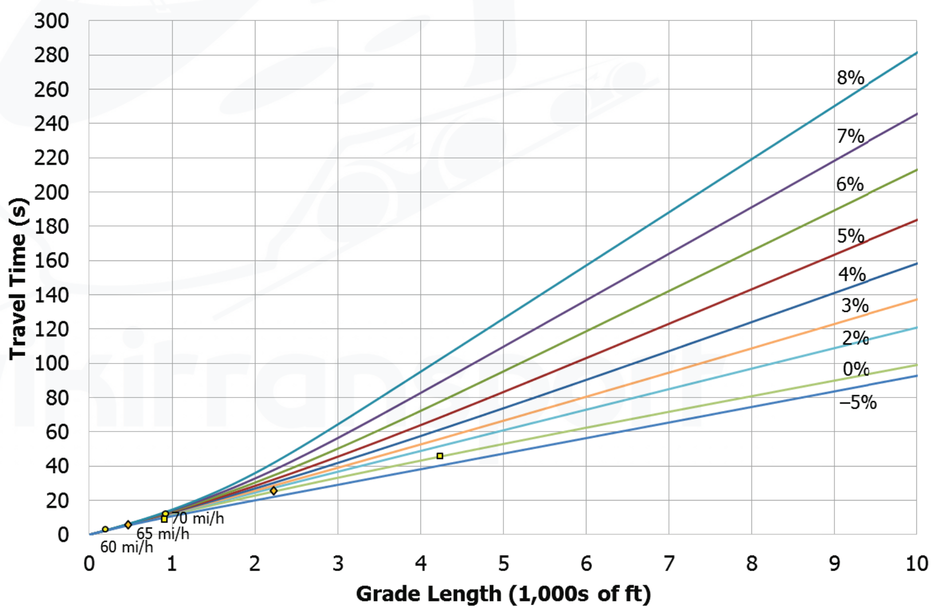
Notes: Curves in this graph assume a weight-to-horsepower ratio of 150.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 25-A15
TT Travel Time Versus Distance Curves for 50-mi/h Initial Speed

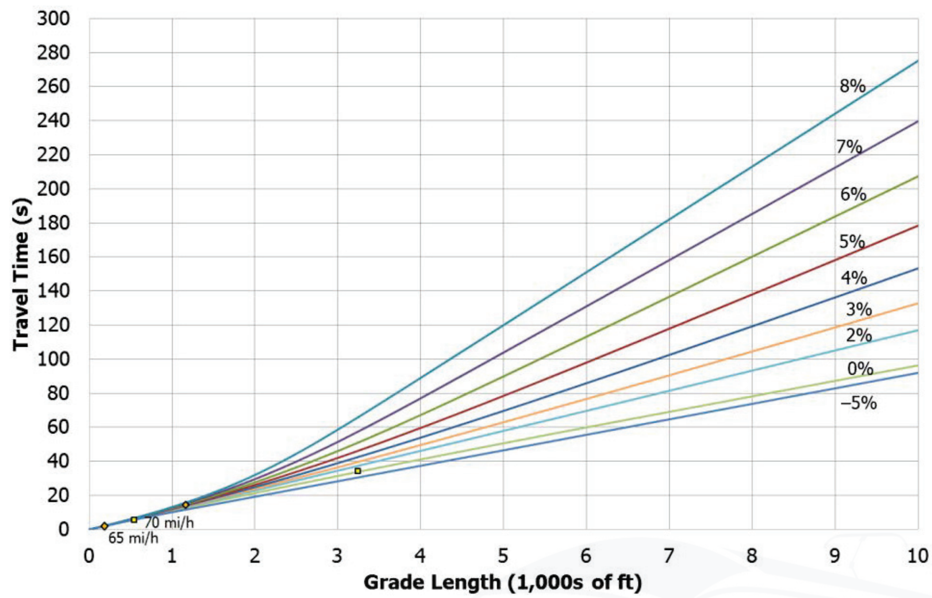


Notes: Curves in this graph assume a weight-to-horsepower ratio of 150.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 25-A16
TT Travel Time Versus Distance Curves for 55-mi/h Initial Speed

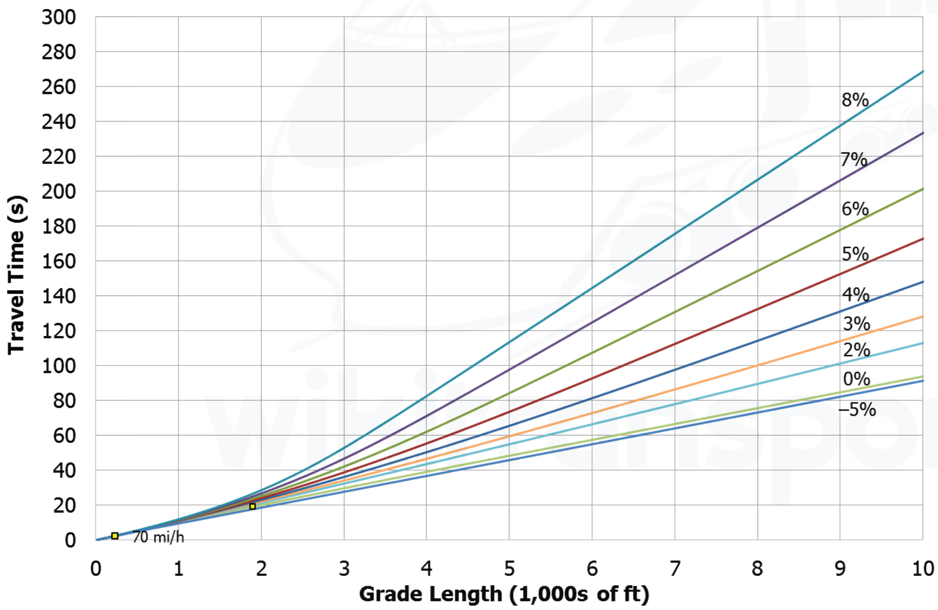


Notes: Curves in this graph assume a weight-to-horsepower ratio of 150.
Circles indicate where a truck reaches 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.



Notes: Curves in this graph assume a weight-to-horsepower ratio of 150.
Diamonds indicate where a truck reaches 65 mi/h and squares indicate 70 mi/h.

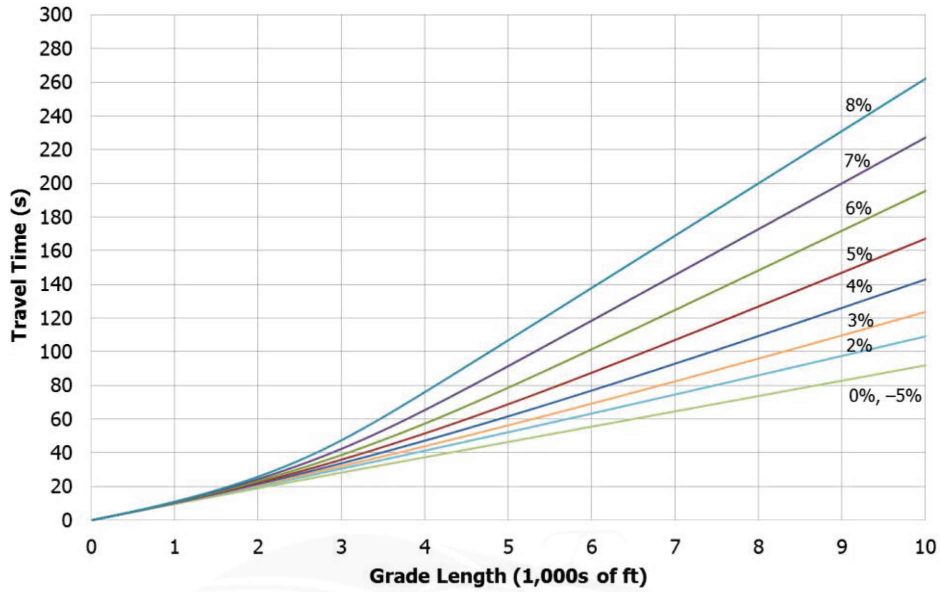
Exhibit 25-A17
TT Travel Time Versus Distance Curves for 60-mi/h Initial Speed



Notes: Curves in this graph assume a weight-to-horsepower ratio of 150.
Squares indicate where a truck reaches 70 mi/h.

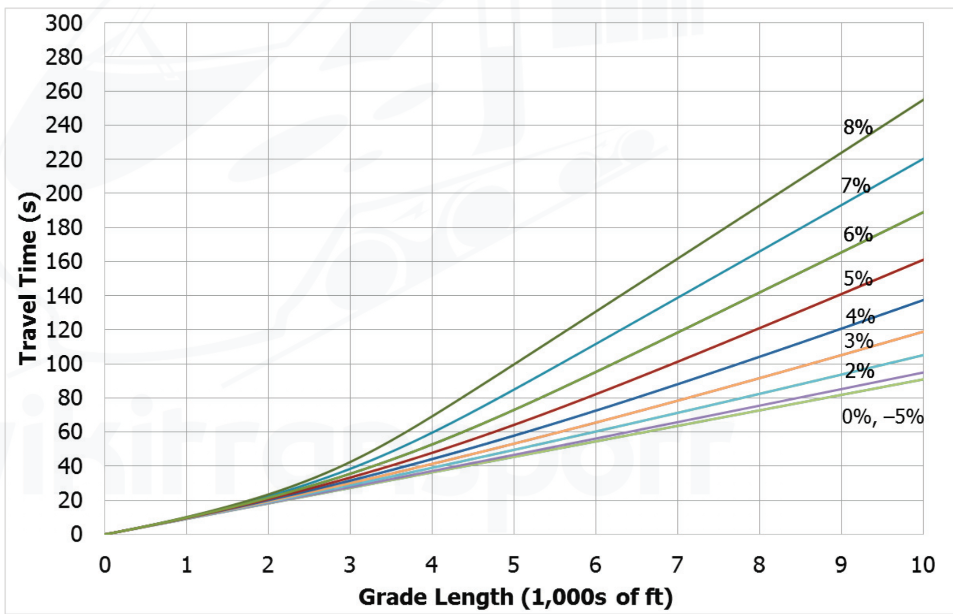
Exhibit 25-A18
TT Travel Time Versus Distance Curves for 65-mi/h Initial Speed

Exhibit 25-A19
TT Travel Time Versus Distance Curves for 70-mi/h Initial Speed



Note: Curves in this graph assume a weight-to-horsepower ratio of 150.

Exhibit 25-A20
TT Travel Time Versus Distance Curves for 75-mi/h Initial Speed



Note: Curves in this graph assume a weight-to-horsepower ratio of 150.