CHAPTER 26
FREEWAY AND HIGHWAY SEGMENTS: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 26 is the supplemental chapter for Chapter 12, Basic Freeway and Multilane Highway Segments, and Chapter 15, Two-Lane Highways, which are found in Volume 2 of the *Highway Capacity Manual* (HCM).

Section 2 provides state-specific heavy-vehicle default values that can be applied to freeway, multilane highway, and two-lane highway analysis.

Section 3 presents a supplemental procedure for basic freeway segments that can be used to assess their operating performance under mixed-flow conditions when significant truck presence, a prolonged single upgrade, or both exist. Appendix A provides travel time versus distance curves for single-unit trucks (SUTs) and tractor-trailers (TTs) for a range of free-flow speeds (FFS) for use with this procedure. Chapter 25, Freeway Facilities: Supplemental, presents an extension of this method for composite grades on freeway facilities.

Section 4 provides suggested capacity and FFS adjustments to account for the effects of different proportions of motorists on a freeway or multilane highway who are not regular users of the facility.

Section 5 presents freeway capacity definitions, guidance on locating sensors for use in measuring freeway capacity, and guidance on estimating capacity from the collected sensor data.

Section 6 provides seven example problems demonstrating the basic freeway and multilane highway segment procedure presented in Chapter 12.

Section 7 provides five example problems demonstrating the motorized vehicle and bicycle methodologies for two-lane highways presented in Chapter 15.

Appendix B describes a methodology for calculating capacity and related performance measures for work zones along two-lane highways that involve the closure of a single lane.

2. STATE-SPECIFIC HEAVY-VEHICLE DEFAULT VALUES

Research into the percentage of heavy vehicles on uninterrupted-flow facilities (*1*) found such a wide range of average values from state to state that not even regional default values could be developed. Exhibit 26-1 presents default values for the percentage of heavy vehicles on freeways by state and area population based on data from the 2004 Highway Performance Monitoring System. Exhibit 26-2 presents similar default values for multilane and two-lane highways. In cases in which states or local jurisdictions have developed their own default values, those values should be used in lieu of the values presented here. Analysts may also wish to develop their own default values based on local or more recent data.

Exhibit 26-1
State-Specific Default Values for Percentage of Heavy Vehicles on Freeways

State	Rural	Small Urban	Medium Urban	Large Urban	State	Rural	Small Urban	Medium Urban	Large Urban
AL[*]	14[a]	7	7	7[a]	MT	22[c]	16[c]	12[c]	NA
AK	4	5[b]	5	3[b]	NC[*]	19[b]	12[b]	12	10[a]
AR	30	24	13	14	ND	21[c]	22[c]	10[c]	NA
AZ	21	19	18	11	NE	36	37	11	8
CA	16	10	7	6	NH	15[b]	12[b]	6[b]	7[b]
CO	12	10	8	7	NJ	8	6	6	9
CT	13	6	6	5	NM	26	12	21	12
DC	NA	NA	NA	4[b]	NV	34[b]	26	18[b]	11[b]
DE	—	—	9[b]	8[b]	NY	18	11	11	7
FL[*]	11	7	12	6	OH	24	13	10	8
GA[*]	19[b]	7[b]	12	8[b]	OK	28	27	12	10
HI	5	19[b]	2	3	OR	26	19	10	7
IA	20[c]	24[c]	11[c]	10[c]	PA	16	13	9	8
ID	29[c]	28[b]	12[b]	7[b]	PR[*]	6	7[b]	7	4[b]
IL	21	23	16	9	RI	3	—	NA	4
IN	26	25	23	14	SC[*]	19[b]	7[b]	7	8[b]
KS	21[c]	17[c]	8[c]	9[b]	SD	20[c]	14[c]	9[c]	NA
KY[*]	20[a]	16	12	10[a]	TN[*]	19	12	12	8
LA[*]	12[c]	7[b]	12	10[c]	TX	16	28[c]	8	5
MA	7[a]	5	4[a]	4	UT	34[c]	—	18	13
MD	18	14	17	8	VA[*]	9	7	7	4
ME	5	5	5	NA	VT	15	12	6	NA
MI	18	12	13	8	WA	11	10	7	6
MN	11	10	6	4	WI	6	6	6	6
MO	29[b]	23[b]	13[b]	10[b]	WV	16[b]	13[b]	9[b]	NA
MS[*]	9[b]	7[b]	7	6[b]	WY	33[c]	36[a]	28[c,d]	NA

Source: Zegeer et al. (*1*).

Notes: Populations are as follows: rural: <5,000; small urban: 5,000–50,000; medium urban: 50,000–250,000; large urban: >250,000.

Values shown represent mean values for the state for each population type except as otherwise noted.

NA = population group does not exist within the state; — = data not available.

* Because of limited data, small urban values were combined for two groups of states: AL, MS, PR, SC, and VA and FL, GA, KY, LA, NC, and TN. Medium urban values were combined for AL, FL, and VA.

[a] Reported values appeared to be a mix of field observations and statewide values. The latter were discounted, such that the averages shown are based primarily on values deemed to be field observations, with some consideration given to nearby states and the value state personnel thought was statewide.

[b] The default value was estimated from field observations from nearby states because of insufficient field data, a lack of data for this road type, or too-heavy reliance on statewide values.

[c] The peak period percentage is identical to the daily average percentage for nearly all observations in the 2004 Highway Performance Monitoring System data set. Default values were estimated primarily from the daily average value but took into account the results from nearby states, particularly the difference between peak and daily values in those states.

[d] This distribution was bimodal, with one group centered on 19% and the other on 44%.

State	Two-Lane Highways		Multilane Highways		State	Two-Lane Highways		Multilane Highways	
	Rural	Small Urban	Rural	Small Urban		Rural	Small Urban	Rural	Small Urban
AL	6[a]	6[a]	4[a]	6[a]	MT	10[c]	4[c]	6[c]	3[c]
AK	10	2	6	3	NC	8[b]	4[b]	6[b]	6[b]
AR	14	7	11	12	ND	14[c]	3[c]	12[c]	7[c]
AZ	9	11	9	9	NE	10	3	12	5
CA	9	5	9	6	NH	6[b]	6[a]	6[b]	6[b]
CO	11	4	5	5	NJ	8	7	8	6[b]
CT	3	3	2	6[b]	NM	17	7	23	12
DC	NA	NA	NA	NA	NV	17[b]	5[c]	10[c]	6[c]
DE	7	6	9	8	NY	8	5	8	5
FL	8	4	7	7	OH	11	4	14	9
GA	8[b]	5[b]	6[b]	6[b]	OK	14[a]	5	17	11
HI	3	3	2	2	OR	12	5	6	9
IA	4[c]	5[c]	5[c]	4[c]	PA	6	3	5	4
ID	12[c]	7[c]	16[c]	9[c]	PR	5	5[b]	5	6
IL	8	5	8	6	RI	2	1	2	6[b]
IN	10	6[a]	12	10	SC	8[b]	5[b]	6[b]	6[b]
KS	15[a]	3	12[c]	6[c]	SD	13[c]	4[c]	12[c]	7[c]
KY	16[a]	6[a]	9[a]	6[a]	TN	5	4[a]	6	4
LA	16[c]	10[c]	6[b]	16	TX	13	9	12	9
MA	3[a]	3[a]	7[b]	6[b]	UT	20[c]	9[c]	22[c]	14[c]
MD	10	6	12	8	VA	4	2	5	2
ME	5	3	4	3	VT	8	5[a]	7	6[b]
MI	9	7[a]	8	4	WA	15	8[a]	10	7
MN	9	8[a]	8	6	WI	4	5[a]	4	5[a]
MO	9[c]	6[c]	12[b]	10[c]	WV	6[b]	6[b]	5[b]	6[b]
MS	14[a]	5[a]	6[b]	6[a]	WY	15[c]	6[c]	10[c]	9[c]

Exhibit 26-2
State-Specific Default Values for Percentage of Heavy Vehicles on Multilane and Two-Lane Highways

Source: Zegeer et al. (*1*).

Notes: Populations are as follows: rural: <5,000; small urban: 5,000–50,000.

Values shown represent mean values for the state for each population type except as otherwise noted.

NA = population group does not exist within the state.

[a] Reported values appeared to be a mix of field observations and statewide values. The latter were discounted, such that the averages shown are based primarily on values deemed to be field observations, with some consideration given to nearby states and the value state personnel thought was statewide.

[b] Either there are insufficient field data, such that regional averages were used, or there are no usable field data, either because there are no data in the state for this road type or because there is a too-heavy reliance on statewide values for both the peak period and the daily average. In these cases, the default value was estimated from field observations for nearby states.

[c] The peak period percentage is identical to the daily average percentage for all or almost all observations in the 2004 Highway Performance Monitoring System data set for this cell. Default values were estimated primarily from the daily average value for this cell, taking into account the results for other similar states in the same region, and in particular the difference between peak and daily average values in those states.

3. TRUCK ANALYSIS USING THE MIXED-FLOW MODEL

INTRODUCTION

This section presents a supplemental procedure that can be used to assess the operating performance of freeway segments under mixed-flow conditions when significant truck presence, a prolonged single upgrade, or both exist. This procedure *must* be used if the analyst is interested in estimating space mean speeds and densities for cars and trucks separately or for the mixed-traffic stream.

Chapter 12, Basic Freeway and Multilane Highway Segments, describes a methodology drawn from this procedure that can be used to assess a segment's level of service (LOS) by converting heavy vehicles into passenger cars by using passenger car equivalent (PCE) values. However, users are cautioned that the auto-only speeds and densities estimated by the PCE-based procedure are likely to be an approximation of reality at high truck percentages and on steep upgrades. For these situations, the mixed-flow model described here is recommended.

Analysts can also use the mixed-flow model for analyzing downgrades and both types of general terrain (level and rolling). When the truck percentage is low or the upgrade is not steep, both the mixed-flow model and the Chapter 12 PCE-based method provide similar results. Chapter 25, Freeway Facilities: Supplemental, extends the mixed-flow model to freeway facilities with multiple, composite grades. National research (2) shows that when the truck presence is low or the upgrade is not steep, both the mixed-flow model and the procedure applying PCE values provide similar results.

OVERVIEW OF THE METHODOLOGY

The process flow for applying the mixed-flow model is depicted in Exhibit 26-3. Selected parameters referenced in the methodology are indicated in Exhibit 26-4 for a 70-mi/h auto-only traffic stream and a representative mixed-traffic stream.
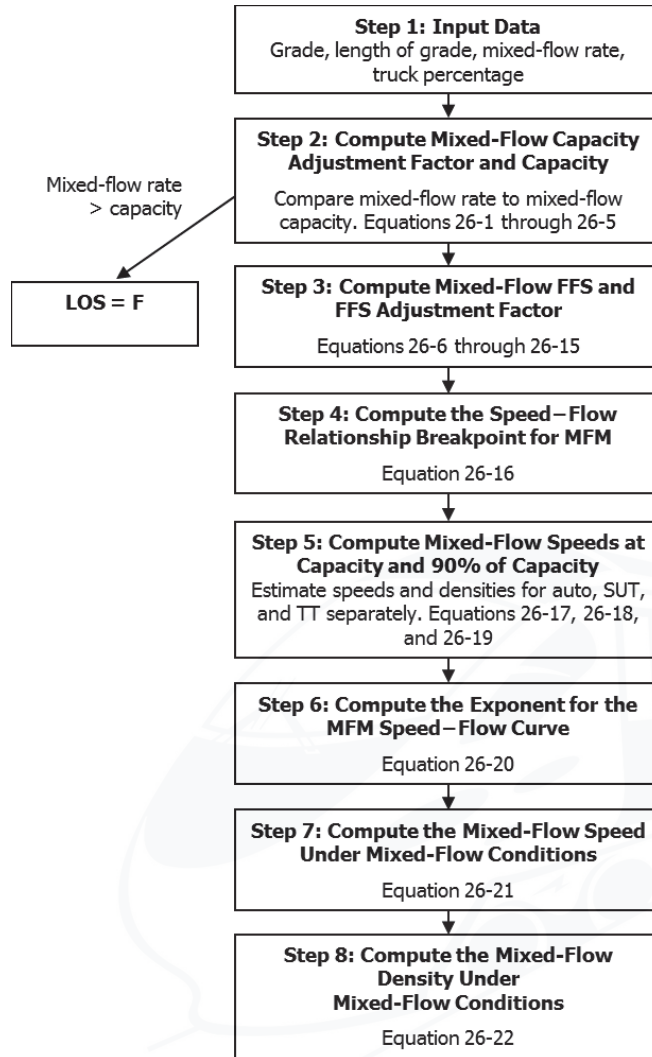
Notes: SUT = single-unit truck; TT = tractor-trailer; FFS = free-flow speed; MFM = mixed-flow model.

Notes: BP = breakpoint; FFS = free-flow speed; c = capacity.

Step 1: Input Data

For a typical operational analysis, the analyst must specify the flow rate of the mixed-traffic stream v_{mix}, grade g, grade length d, SUT percentage P_{SUT}, and TT percentage P_{TT} for the traffic stream.

Step 2: Compute Mixed-Flow Capacity Adjustment Factor and Capacity

The capacity adjustment factor (CAF) for mixed-flow CAF_{mix} converts auto-only capacities into mixed-traffic capacities. It is computed with Equation 26-1.

Equation 26-1

$$CAF_{mix} = CAF_{ao} - CAF_{T,mix} - CAF_{g,mix}$$

where

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

CAF_{ao} = capacity adjustment factor for the auto-only case (decimal);

$CAF_{T,mix}$ = capacity adjustment factor for percentage of trucks for the mixed-flow case (decimal); and

$CAF_{g,mix}$ = capacity adjustment factor for grade for the mixed-flow case (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 $CAF_{T,mix}$ is computed with Equation 26-2.

Equation 26-2

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

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 $CAF_{g,mix}$ accounts for the grade severity, grade length, and truck presence. It is computed by using Equation 26-3 with Equation 26-3.

Equation 26-3

$$CAF_{g,mix} = \rho_{g,mix} \times \max[0, 0.69 \times (e^{12.9g} - 1)]$$
$$\times \max[0, 1.72 \times (1 - 1.71e^{-3.16d})]$$

with

Equation 26-4

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

where

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

P_T = total truck percentage (decimal),

g = grade (decimal), and

d = grade length (mi).

Once CAF_{mix} is computed, the mixed-flow capacity can be computed with Equation 26-5.

$$C_{mix} = C_{ao} \times CAF_{mix}$$

where

C_{mix} = mixed-flow capacity (veh/h/ln);

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

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

If the input flow rate of the mixed-traffic stream v_{mix} exceeds the mixed-flow capacity computed in Equation 26-5, then LOS F prevails, and the segment procedure stops. A facility analysis is recommended under these conditions.

Step 3: Compute Mixed-Flow FFS and FFS Adjustment Factor

Equation 26-6 through Equation 26-8 compute the free-flow travel rates (in seconds per mile) for SUTs, TTs, and automobiles, respectively, for a specific segment with a steep grade, high truck percentage, or both. For the purposes of calculating the automobile free-flow travel rate, the flow rate of the mixed-traffic stream v_{mix} is assumed to be 1 veh/h/ln when Equation 26-8 is used.

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

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

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

where

τ_a = automobile free-flow travel rate (s/mi),

τ_{SUT} = SUT free-flow travel rate (s/mi),

τ_{TT} = TT free-flow travel rate (s/mi),

$\tau_{SUT,kin}$ = kinematic travel rate of SUTs (s/mi),

$\tau_{TT,kin}$ = kinematic travel 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} = SUT percentage (decimal),

P_{TT} = TT percentage (decimal), and

3,600 = number of seconds in 1 h.

Traffic Interaction Term

The traffic interaction term computed by Equation 26-9 is the contribution of traffic interactions to mixed-flow FFS. For the purposes of calculating the automobile free-flow travel rate, the traffic interaction term $\Delta\tau_{TI}$ is set to 0 when Equation 26-8 is used.

Equation 26-9

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

where

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

S_{ao} = auto-only speed for the given flow rate, from Equation 26-10 (mi/h);

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

CAF_{mix} = mixed-flow capacity adjustment factor for the basic freeway segment from Equation 26-1 (decimal).

Auto-Only Speed for the Given Flow Rate

The auto-only travel rate for the given flow rate is computed with Equation 26-10.

Equation 26-10

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

where

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

c = base segment capacity, from Exhibit 12-6 (pc/h/ln);

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

D_c = density at capacity = 45 pc/mi/ln; and

CAF_{mix} = mixed-flow capacity adjustment factor for the basic freeway segment, from Equation 26-1 (decimal).

Kinematic Travel Rates for SUTs and TTs

The kinematic travel rates for SUTs and TTs are obtained from truck travel time versus distance performance curves on the basis of the truck weight-to-horsepower ratio, grade, and grade length. Exhibit 26-5 shows truck travel time versus distance curves for a representative SUT starting from a speed of 70 mi/h. Alternate representations of how the propulsive and resistive forces vary with speed can produce slightly different results (e.g., *3*, *4*).

Exhibit 26-6 shows the corresponding curves for TTs for a base FFS of 70 mi/h. These curves can be used when the base FFS is within 2.5 mi/h of 70 mi/h. Appendix A provides additional curves for SUTs and TTs for FFS values of 50, 55, 60, 65, and 75 mi/h.

On downgrades, trucks are able to maintain their FFS, and their kinematic performance is the same as passenger cars. The analyst could use the Chapter 12 PCE-based method instead of the mixed-flow model in those cases.
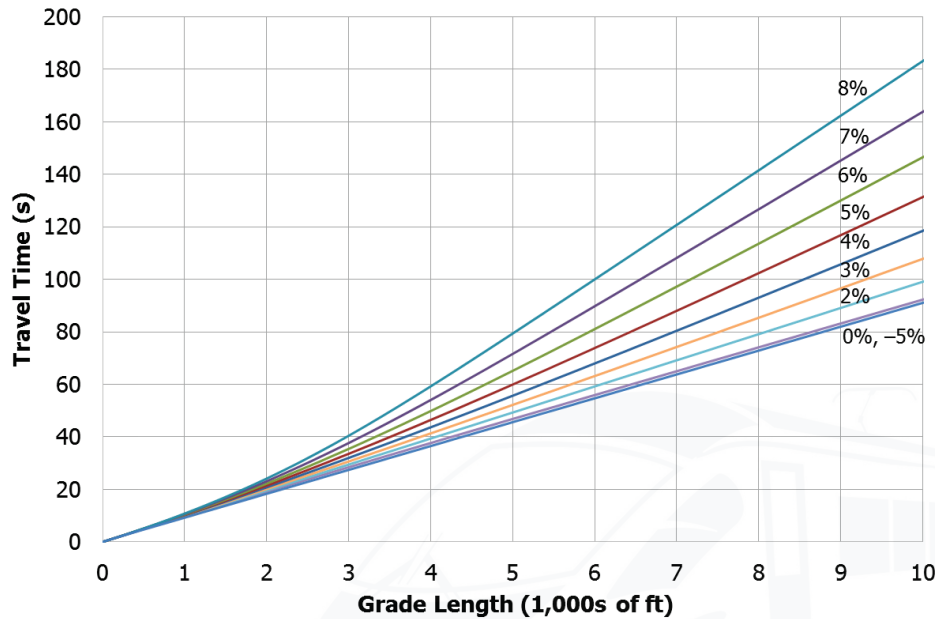
Exhibit 26-5
SUT Travel Time Versus Distance Curves for 70-mi/h FFS

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

Exhibit 26-6
TT Travel Time Versus Distance Curves for 70-mi/h FFS
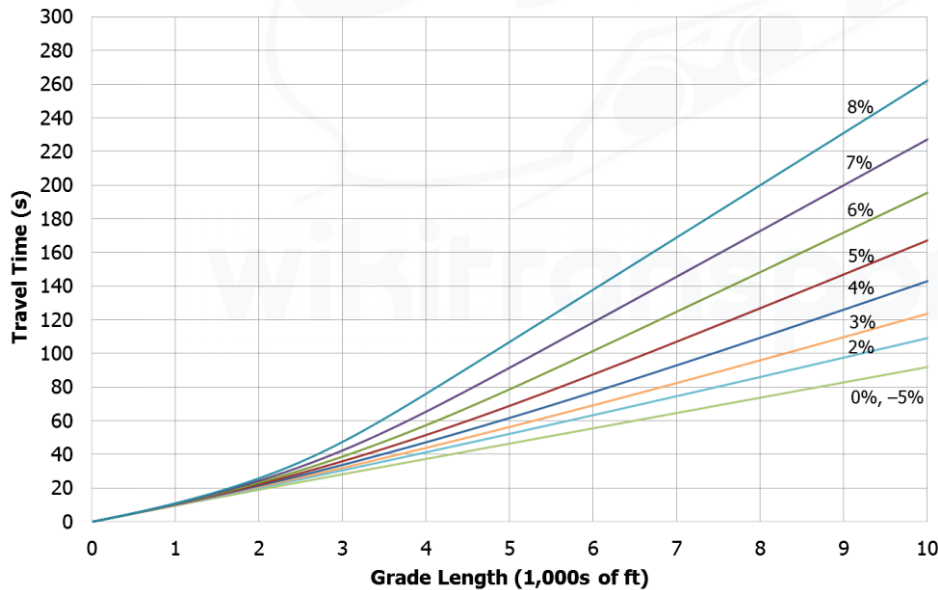
Note: Curves in this graph assume a weight-to-horsepower ratio of 150.

The *x*-axis in Exhibit 26-5 and Exhibit 26-6 represents the distance *d* traveled by the truck, and the *y*-axis represents the travel time *T* to cover the grade length *d*. Different curves provide the travel times for different upgrades. The kinematic space mean travel rate can be computed with Equation 26-11.

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

Equation 26-11

where

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

T = travel time (s), and

d = grade length (mi).

The maximum grade length shown in Exhibit 26-5 and Exhibit 26-6 is 10,000 ft. When the grade is longer than 10,000 ft, the kinematic travel rate can be computed with Equation 26-12.

Equation 26-12

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

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 value for SUTs and TTs is shown in Exhibit 26-7 and Exhibit 26-8, respectively, for different combinations of grade and FFS.

Exhibit 26-7
δ Values for SUTs

Grade	Free-Flow Speed (mi/h)					
	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 26-8
δ Values for TTs

Grade	Free-Flow Speed (mi/h)					
	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 $\tau_{SUT,kin}$ and $\tau_{TT,kin}$ are obtained, Equation 26-6 and Equation 26-7 can be used to add the traffic interaction term to obtain the truck free-flow travel rates τ_{SUT} and τ_{TT}. Equation 26-8 can then be used to compute the automobile free-flow travel rate τ_a. Again, the mixed-flow rate v_{mix} is assumed to be 1 veh/h/ln when Equation 26-8 is used to estimate the automobile free-flow travel rate.

Mixed-Flow FFS

Equation 26-13 converts individual free-flow travel rates by mode into a mixed-flow free-flow travel rate, and Equation 26-14 then converts the mixed-flow free-flow travel rate into a mixed-flow FFS.

$$\tau = P_a\tau_a + P_{SUT}\tau_{SUT} + P_{TT}\tau_{TT}$$

Equation 26-13

$$FFS_{\text{mix}} = \frac{3{,}600}{\tau} = \frac{3{,}600}{P_a\tau_a + P_{SUT}\tau_{SUT} + P_{TT}\tau_{TT}}$$

Equation 26-14

where

τ = mixed-flow free-flow travel rate (s/mi),

τ_a = automobile free-flow travel rate (s/mi),

τ_{SUT} = SUT free-flow travel rate (s/mi),

τ_{TT} = TT free-flow travel rate (s/mi),

P_a = automobile percentage (decimal),

P_{SUT} = SUT percentage (decimal),

P_{TT} = TT percentage (decimal), and

FFS_{mix} = mixed-flow free-flow speed (mi/h).

FFS Adjustment Factor

The segment's speed adjustment factor (SAF) is estimated with Equation 26-15.

$$SAF_{\text{mix}} = FFS_{\text{mix}}/FFS$$

Equation 26-15

where

SAF_{mix} = mixed-flow speed adjustment factor for the basic freeway segment (decimal),

FFS_{mix} = mixed-flow free-flow speed (mi/h), and

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

Step 4: Compute the Speed–Flow Relationship Breakpoint for the Mixed-Flow Model

The breakpoint is the maximum flow rate up to which speed is maintained at the adjusted FFS level. It is computed by Equation 26-16 and is depicted in Exhibit 26-4.

$$BP_{\text{mix}} = \max\left[0, BP_{ao}(1 - 0.4P_T{}^{0.1} \times \max[0, e^{30g} + 1] \times d^{0.01})\right]$$

Equation 26-16

where

BP_{mix} = breakpoint for mixed flow (veh/h/ln);

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

P_T = total truck percentage (decimal);

g = grade (decimal); and

d = grade length (mi).

Step 5: Compute Mixed-Flow Speeds at Capacity and 90% of Capacity

To determine the mixed-flow speeds for the given mixed-flow rate, mixed-flow speeds at capacity and 90% of capacity are computed for calibration purposes. This computation, in turn, requires applying Equation 26-6 through Equation 26-8 to calculate individual speeds for SUTs, TTs, and automobiles, respectively. The equations are applied twice, first applying the value of C_{mix} as v_{mix} to calculate speed at capacity, and then applying the value of $0.9C_{mix}$ as v_{mix} to calculate speed at 90% of capacity.

The resulting modal travel time rates are converted to modal speeds S_m by using Equation 26-17.

Equation 26-17

$$S_m = \frac{3{,}600}{\tau_m}$$

where S_m is the speed (mi/h) for mode m (SUT, TT, or automobile), and τ_m is the travel time rate (s/mi) for mode m.

Next, densities for individual modes are computed with Equation 26-18.

Equation 26-18

$$D_m = v_m/S_m$$

where D_m is the density (SUT/mi, TT/mi, or pc/mi, depending on the mode) for mode m, v_m is the flow rate (SUT/h, TT/h, or pc/h) for mode m, and S_m is the speed (mi/h) for mode m.

Finally, the mixed-flow speed used for calibration S_{calib} is calculated with Equation 26-19.

Equation 26-19

$$S_{calib} = \frac{3{,}600}{P_a\tau_a + P_{SUT}\tau_{SUT} + P_{TT}\tau_{TT}}$$

Equation 26-19 is applied twice (i.e., two calibration points are needed), once using τ values at capacity and again using τ values for 90% of capacity.

Mixed-flow travel rates and mixed-flow speeds are calculated with Equations 26-13 and 26-14 twice (i.e., two calibration points are needed), once at capacity and once at 90% capacity.

Step 6: Compute the Exponent for the Mixed-Flow Model Speed–Flow Curve

The exponent for the speed–flow curve, which describes the rate at which speed drops as the flow rate increases in the nonlinear portion of the mixed-flow speed–flow curve (see Exhibit 26-4), is computed with Equation 26-20.

Equation 26-20

$$\phi_{mix} = 1.195 \times \frac{\ln\left(\frac{FFS_{mix} - S_{calib,90cap}}{FFS_{mix} - S_{calib,cap}}\right)}{\ln\left(\frac{0.9C_{mix} - BP_{mix}}{C_{mix} - BP_{mix}}\right)}$$

where

φ_{mix} = exponent for the speed–flow curve (decimal),

FFS_{mix} = mixed-flow free-flow speed (mi/h),

$S_{calib,90cap}$ = mixed-flow speed at 90% of capacity (mi/h),

$S_{calib,cap}$ = mixed-flow speed at capacity (mi/h),

C_{mix} = mixed-flow capacity (veh/h/ln), and

BP_{mix} = breakpoint for mixed flow (veh/h/ln).

Step 7: Compute the Mixed-Flow Speed Under Mixed-Flow Conditions

The mixed-flow speed for mixed-flow conditions is computed by using the generic form of the basic freeway segment speed–flow model, as shown in Equation 26-21.

$$S_{mix} = \begin{cases} FFS_{mix} & v_{mix} \le BP_{mix} \\ FFS_{mix} - \left(FFS_{mix} - S_{calib,cap}\right)\left(\dfrac{v_{mix} - BP_{mix}}{C_{mix} - BP_{mix}}\right)^{\phi_{mix}} & v_{mix} > BP_{mix} \end{cases}$$

Equation 26-21

where

S_{mix} = mixed-flow speed (mi/h),

FFS_{mix} = mixed-flow free-flow speed (mi/h),

$S_{calib,cap}$ = mixed-flow speed at capacity (mi/h),

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

BP_{mix} = breakpoint for mixed flow (veh/h/ln),

C_{mix} = mixed-flow capacity (veh/h/ln), and

φ_{mix} = exponent for the speed–flow curve (decimal).

Step 8: Compute the Mixed-Flow Density Under Mixed-Flow Conditions

The mixed-flow density is computed by Equation 26-22.

$$D_{mix} = v_{mix}/S_{mix}$$

Equation 26-22

where

D_{mix} = mixed-flow density (veh/mi/ln),

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

S_{mix} = mixed-flow speed (mi/h).

4. ADJUSTMENTS FOR DRIVER POPULATION EFFECTS

The base traffic stream characteristics for basic freeway and multilane highway segments are representative of traffic streams composed primarily of commuters or drivers who are familiar with the facility. It is generally accepted that traffic streams with different characteristics (e.g., recreational trips) use freeways less efficiently. Although data are sparse and reported results vary substantially, significantly lower capacities have been reported on weekends, particularly in recreational areas. Thus, it may generally be assumed the reduction in capacity extends to service flow rates and service volumes for other levels of service as well. In addition, it is expected that a reduction in FFS would be observed when large numbers of unfamiliar drivers are present in a freeway or multilane highway traffic stream.

The driver population adjustment factor f_p has previously been used in the HCM to reflect the effects of unfamiliar drivers in the traffic stream; it was applied as an increase in demand volume. The values of f_p ranged from 0.85 to 1.00 in most cases, although lower values have been observed in isolated cases. The HCM recommended the analyst use a value of 1.00 for this factor (reflecting a traffic stream composed of commuters or other regular drivers), unless there was sufficient evidence that a lower value should be used. When greater accuracy was needed, comparative field studies of commuter and noncommuter traffic flow and speeds were recommended.

With the addition of a unified speed–flow equation in Chapter 12, Basic Freeway and Multilane Highway Segments, and the ability to adjust both the base FFS and capacity in all freeway segment chapters (Chapters 12, 13, and 14) to account for incidents and weather events, the driver population factor is no longer used. Instead, FFS and capacity adjustment factors SAF_{pop} and CAF_{pop} are applied in combination with other applicable SAFs and CAFs.

In the absence of new research on driver population effects, recommended values of SAF_{pop} and CAF_{pop} have been developed that produce similar density results as those predicted using the former driver population factor approach. This conversion was performed by using the unified equation of Chapter 12 and therefore represents a slight approximation in the cases of weaving, merge, and diverge segments.

Judgment is still required when the analyst applies these adjustments and, in the absence of information to the contrary, the default value for SAF_{pop} and CAF_{pop} is always 1.0. Should the analyst expect a significant presence of unfamiliar drivers, the values shown in Exhibit 26-9 can serve as a guide for the analysis.

Exhibit 26-9
Recommended CAF and SAF Adjustments for Driver Population Impacts

Level of Driver Familiarity	CAF_{pop}	SAF_{pop}
All familiar drivers, regular commuters	1.000	1.000
Mostly familiar drivers	0.968	0.975
Balanced mix of familiar and unfamiliar drivers	0.939	0.950
Mostly unfamiliar drivers	0.898	0.913
All or overwhelmingly unfamiliar drivers	0.852	0.863

5. GUIDANCE FOR FREEWAY CAPACITY ESTIMATION

This section presents guidance for field measuring and estimating freeway capacity. The section is organized as follows: overall definitions of freeway capacity, guidance for field data collection using sensors, and guidance for estimating capacity from the collected data.

FREEWAY CAPACITY DEFINITIONS

Freeway segment capacity is commonly understood to be a maximum flow rate that is associated with the occurrence of some type of breakdown that in turn results in lower speeds and higher densities after the breakdown event. When oversaturation begins, queues develop and vehicles discharge from the bottleneck at a queue discharge rate that is usually lower than the throughput rate before the breakdown. This lower discharge rate after a breakdown is also known as the *capacity drop phenomenon.* Several key terms related to freeway capacity are defined below as they apply to this chapter.

Freeway Breakdown

A flow breakdown on a freeway represents the transition from uncongested to congested conditions, as evidenced by the formation of queues upstream of the bottleneck and reduced prevailing speeds.

In the HCM freeway methodology, the breakdown event on a freeway bottleneck is defined as a sudden drop in speed at least 25% below the FFS for a sustained period of at least 15 min that results in queuing upstream of the bottleneck.

Recovery

A freeway segment is considered to have recovered from the breakdown event and the resulting oversaturated conditions when the average speed (or density) reaches prebreakdown conditions for a minimum duration of 15 min. The definition of recovery is therefore the inverse of the definition of breakdown, requiring a recovery to be near prebreakdown conditions (operations above the speed threshold) for at least 15 min.

The HCM defines the breakdown recovery on a freeway bottleneck as a return of the prevailing speed to within 10% of the FFS for a sustained period of at least 15 min, without the presence of queuing upstream of the bottleneck.

Prebreakdown Flow Rate

The prebreakdown flow rate is the flow rate that immediately precedes the occurrence of a breakdown event. The literature suggests this flow rate does not have a fixed value, as evidence shows breakdowns are stochastic in nature and can occur following a range of flow rates. The prebreakdown flow rate is typically expressed in units of passenger cars per hour per lane. To achieve a uniform expression, trucks and other heavy vehicles are converted into an equivalent passenger car traffic stream.

In the HCM, the prebreakdown flow rate is defined as the 15-min average flow rate that occurs immediately prior to the breakdown event. For the purposes of this chapter, the prebreakdown flow rate is equivalent to the segment capacity.

Postbreakdown Flow Rate or Queue Discharge Flow Rate

The postbreakdown flow rate is also referred to as the *queue discharge flow rate* or the average discharge flow rate. This flow rate is usually lower than the prebreakdown flow rate, resulting in a significant loss of freeway throughput during congestion. Cases in which the postbreakdown flow rate exceeds the prebreakdown flow rate have been observed, mostly when the prebreakdown flow rate is low. Studies (*5*) have indicated the average difference between postbreakdown and prebreakdown flow rates varies widely, from as little as 2% to as much as 20%. In the absence of local information, a default value of 7% is recommended.

In the HCM, the queue discharge flow rate is defined as the average flow rate during oversaturated conditions (i.e., during the time interval after breakdown and prior to recovery).

CAPACITY MEASUREMENT LOCATIONS

Research at freeway merging segments (*6*) has found a breakdown may first be observed either upstream or downstream of the actual bottleneck. Some research has indicated a breakdown may first be observed upstream of the bottleneck, slowly spreading downstream as vehicles accelerate past the start of the bottleneck. Other research has found the breakdown initially occurs downstream of the merge point and then moves upstream as a shock wave.

To identify the breakdown event from field data, the following process should be followed:

- Data are obtained at three sensors: (*a*) a bottleneck location (e.g., just downstream of the end of the acceleration lane), (*b*) at a nearby sensor location downstream of the bottleneck, and (*c*) at a nearby sensor location upstream of the bottleneck.
- Upstream and downstream sensors should be within 0.5 mi of the bottleneck, and the freeway ideally should have no entry or exit points between the three sensors (other than, for example, a bottleneck on-ramp).
- The bottleneck detector should be upstream of the beginning of the deceleration lane or downstream of the end of the acceleration lane to avoid missing flow in those lanes.
- The analyst evaluates data from the bottleneck sensor to identify a breakdown by using the definitions provided above.
- The analyst evaluates data from the downstream sensor for the same time period to ensure no breakdown exists, which indicates congestion at the bottleneck sensor is unlikely due to spillback from downstream congestion.

- The analyst evaluates data from the upstream sensor to verify queues are forming as a result of breakdown at the bottleneck. This check ensures observed drops in speeds and increases in density at the bottleneck sensor are indeed due to breakdown.

It is important that the measurements of flows, speeds, and densities used to estimate capacity are carried out at the correct locations, especially if the data will be generated from existing fixed freeway sensors, which may or may not be at the optimal locations to detect breakdown events. Capacity should always be measured at the bottleneck location. At merge bottlenecks or lane drops, this location is downstream of the merge point (Exhibit 26-10). At diverge bottlenecks, this location is upstream of the diverge point (Exhibit 26-11). At weaving bottlenecks, this location is within the weaving area (Exhibit 26-12).
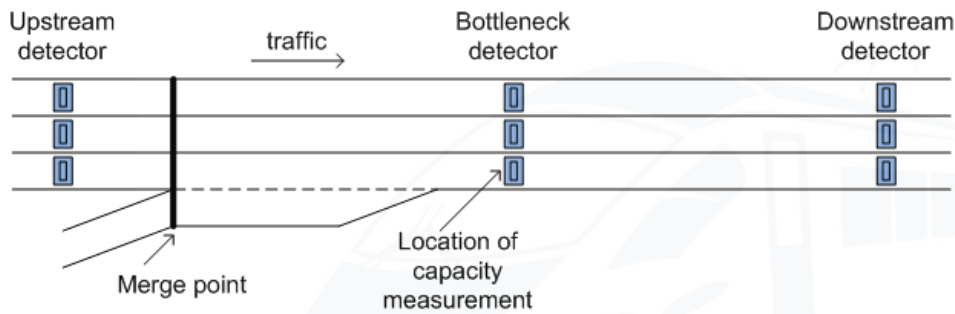
Source: Elefteriadou, Kondyli, and St. George (*6*).
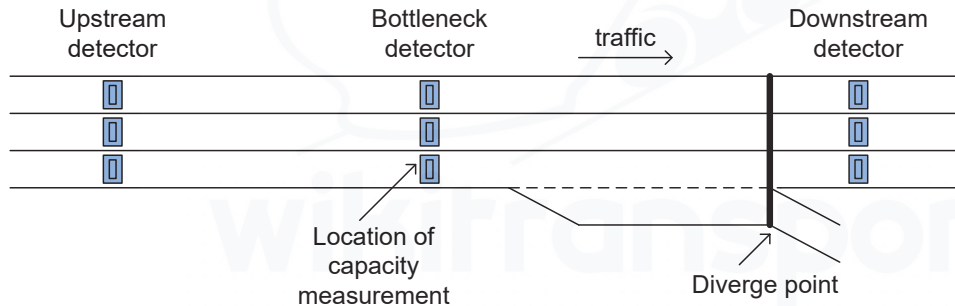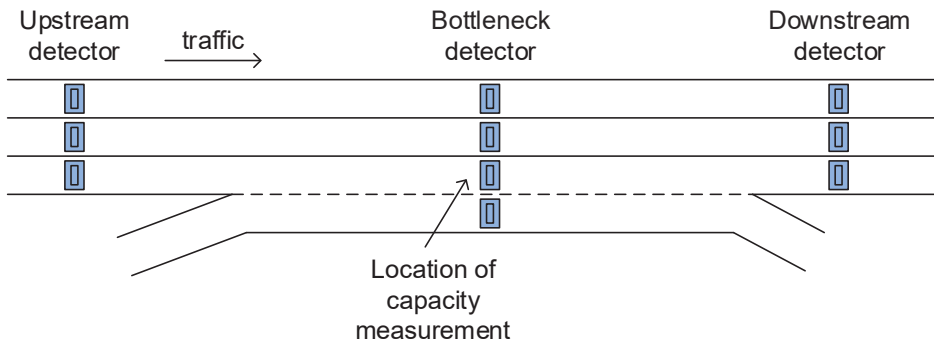
Source: Elefteriadou, Kondyli, and St. George (*6*).

Source: Elefteriadou, Kondyli, and St. George (*6*).

Regardless of the bottleneck type, the analyst will be able to identify and measure capacity only if a breakdown occurs. As discussed below, the breakdown event is associated with the development of queues that form upstream of the bottleneck location (i.e., merge point, diverge point, weaving section) and propagate further upstream, but queues also propagate downstream as vehicles accelerate past the start of the bottleneck. Once breakdown events are identified, the analyst will be able to identify the prebreakdown and postbreakdown flow rates and estimate segment capacity based on the method discussed in the next section.

CAPACITY ESTIMATION FROM FIELD DATA

To estimate the capacity of the various freeway segments it is important to analyze data obtained under *recurring* congestion and under similar operational and weather conditions. Observations in which adverse weather, incidents, work zones, or special events were present must be analyzed separately to obtain capacities under various prevailing conditions. To obtain a reasonable capacity estimate, it is important to analyze a considerable amount of data over a period of several months to an entire year.

The recommended method for capacity estimation from sensor data takes into account that capacity is stochastic. That is, the same flow rate may or may not be followed by a breakdown. Therefore, during an observation period, both prebreakdown flow rates and flow rates that are not followed by breakdown events (uncongested flow rates) are considered. From these flow rates, the method develops a capacity distribution and then selects a capacity value based on an acceptable rate of breakdown. Two plausible (and equivalent) freeway segment capacity definitions are offered:

1. **Definition A:** Freeway segment capacity is the maximum 15-min flow rate (in passenger cars per hour per lane) that produces an acceptable (λ%) rate of breakdown.

2. **Definition B:** Freeway segment capacity is the maximum 15-min flow rate (in passenger cars per hour per lane) that ensures stable flow (100 − λ%) of the time.

The rate of breakdown λ is the ratio of the total number of periods observed under prebreakdown conditions, divided by the total number of 15-min uncongested observations under the same flow rate. A default acceptable rate of breakdown λ of 15% is recommended.

The capacity estimation process follows a series of eight steps and assumes sensors are placed at the appropriate locations (as discussed above) and are available to measure prebreakdown flows and ensure the absence of downstream congestion, which may bias the results described below.

1. Record the distribution of 15-min flow rates (in passenger cars per hour per lane) during the observation period (preferably a long period). For example, sampling from the sensor 24 h per day on weekdays over a year gives approximately $24 \times 4 \times 250 = 24{,}000$ flow rate observations.

2. Exclude the 15-min time periods when the freeway is in breakdown mode, as defined earlier, which will result in a distribution of uncongested 15-min flow rates. It is recommended to filter breakdowns due to nonrecurring sources of congestion, such as severe weather events or incidents, as the focus is on estimating the bottleneck's capacity under recurring congestion conditions.

3. Bin the uncongested flow rates into 100- or 200-pc/h/ln bins.

4. Compute the average flow rate in each bin.

5. For each bin, count the number of times the flow rates in the bin were immediately followed by the occurrence of a breakdown. In other words, bin the prebreakdown 15-min flow rates.

6. Calculate the actual probability of breakdown $P(B_F)$ in each bin, defined as the number of times a flow rate bin was in a prebreakdown condition $n(B)$, divided by the number of times that bin was observed to have occurred, or $n(F)$. The probability of breakdown $P(B_F)$ in each bin is simply $P(B_F) = n(B)/n(F)$.

7. Fit a Weibull distribution (7) to the empirical probability of breakdown computed in Step 6.

8. Based on the selected threshold breakdown (or stable flow) rate λ or $(1 - \lambda)$, determine the resulting capacity value from the Weibull distribution developed in Step 6 by using Equation 26-23. A value of λ of 15% is recommended.

$$\text{Capacity} = \beta \times \sqrt[\gamma]{-\ln(1 - \lambda)}$$

Equation 26-23

where β and γ, respectively, are the shape and scale parameters of the fitted Weibull distribution, and λ is as defined previously. When $\lambda = 0.15$, the equation simplifies to $c = \beta \, (0.163)^{1/\gamma}$.

The following example is based on actual data and involves estimating the capacity of a bottleneck on southbound I-440 in Raleigh, North Carolina. In this example, sensor data in the vicinity of an on-ramp bottleneck were collected for 260 weekdays from June 2014 to May 2015. The average percentage of trucks observed in the traffic stream was less than 1%; therefore, the conversion of trucks into PCEs is ignored for the purposes of this example.

The theoretical number of 15-min observations is 260 days × 96 observations per day = 24,960 observations. After outliers were removed (observations from incident and weather events and congested-flow periods), there remained 22,984 periods when flow was deemed uncongested and that represented similar operational and weather conditions. Within these periods, 192 breakdowns were identified that met the criteria described above.

Exhibit 26-13 summarizes the computations for this example, using the eight steps given above. The example illustrates how the process yields a capacity value based on the recommended 15% breakdown rate.

Exhibit 26-13
Illustrative Example of the
Capacity Estimation Procedure

[1]	[2]	[3]	[4]	[5]	[6]	[7]
		Average	No. of Observed 15-min	No. of Observed 15-min Periods at a	Probability of Breakdown	Cumulative Probability of
Flow Rate in Bins (pc/h/ln)		Flow Rate in Bin	Uncongested	Prebreakdown	Breakdown	Breakdown
From	To	(pc/h/ln)	Periods	Flow Rate	in Bin	
0	99	50	4,570	0	0.0%	0.0%
100	199	150	1,657	1	0.1%	0.5%
200	299	250	1,009	3	0.3%	2.1%
300	399	350	765	2	0.3%	3.1%
400	499	450	889	2	0.2%	4.2%
500	599	550	913	0	0.0%	4.2%
600	699	650	746	0	0.0%	4.2%
700	799	750	657	0	0.0%	4.2%
800	899	850	534	0	0.0%	4.2%
900	999	950	458	0	0.0%	4.2%
1,000	1,099	1,050	798	0	0.0%	4.2%
1,100	1,199	1,150	1,801	1	0.1%	4.7%
1,200	1,299	1,250	2,171	2	0.1%	5.7%
1,300	1,399	1,350	1,662	5	0.3%	8.3%
1,400	1,499	1,450	1,185	8	0.7%	12.5%
1,500	1,599	1,550	866	10	1.2%	17.7%
1,600	1,699	1,650	618	13	2.1%	24.5%
1,700	1,799	1,750	495	22	4.4%	35.9%
1,800	1,899	1,850	322	6	1.9%	39.1%
1,900	1,999	1,950	258	16	6.2%	47.4%
2,000	2,099	2,050	301	45	15.0%	70.8%
2,100	2,199	2,150	227	37	16.3%	90.1%
2,200	2,299	2,250	79	18	22.8%	99.5%
2,300	2,399	2,350	3	1	33.3%	100.0%
2,400	2,499	2,450	0	0	NA	100.0%
Sum			**22,984**	**192**		

Notes: Numbers in brackets indicate column numbers. NA = not applicable.

The exhibit shows 22,984 15-min flow rate observations in Column 4, equivalent to 5,746 h of observations. Column 5 shows 192 breakdown events. The probability of breakdown in a bin is computed in Column 6, which is used to estimate capacity based on the defined λ threshold. Finally, Column 7 shows the cumulative distribution of prebreakdown flow rates, based on the data in Column 5.

The information in Exhibit 26-13 is shown graphically in Exhibit 26-14. The solid black curve to the right shows the Weibull distribution fitted to the data in Column 6; the actual data are also plotted. The distribution parameters were $\beta =$ 2,569 and $\gamma = 9.13$. Substituting these values into Equation 26-23 and using $\lambda =$ 0.15 yields a capacity value of 2,105 pc/h/ln. The gray dashed curve to the left in the exhibit represents the cumulative distribution of prebreakdown flow rates (i.e., Column 7). In this case, the calculated capacity value corresponded to approximately the 85th percentile of the prebreakdown flow rate distribution, as represented by the dotted lines.
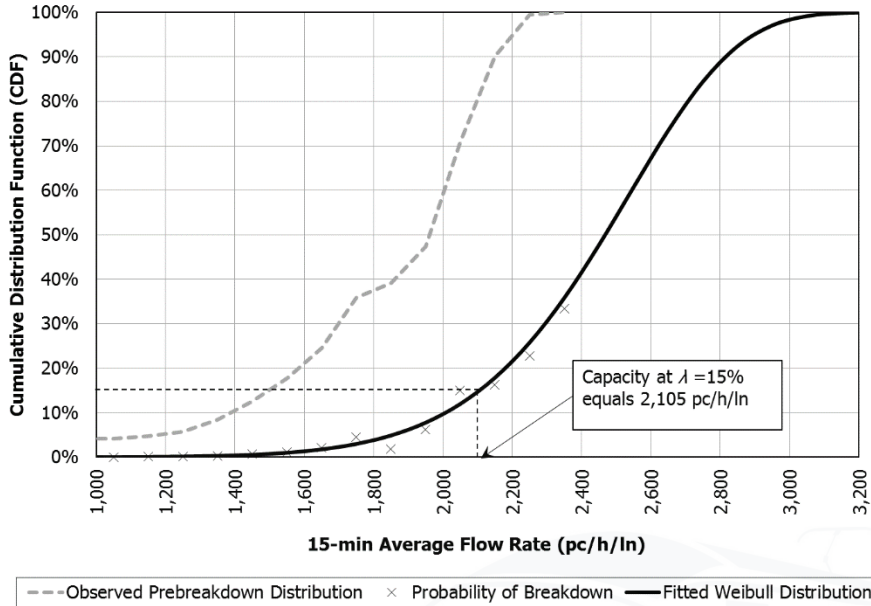
In summary, the capacity estimation method considers the fact that flow rates preceding breakdown can also occur at other times without being followed by a breakdown. The definition of capacity is clear and unambiguous and can be explained to the HCM user or practitioner without much difficulty. However, the analyst needs to ensure there are a *sufficient number of breakdown observations* to be confident in the calculated capacity value.

6. FREEWAY AND MULTILANE HIGHWAY EXAMPLE PROBLEMS

Exhibit 26-15 lists the seven example problems provided in this section. The problems demonstrate the computational steps involved in applying the automobile methodology to basic freeway and multilane highway segments. All the freeway example problems address urban freeway situations.

Example Problem	Description	Application
1	Four-lane freeway LOS	Operational analysis
2	Number of lanes required for target LOS	Design analysis
3	Six-lane freeway LOS and capacity	Operational and planning analysis
4	LOS on a five-lane highway with a two-way left-turn lane	Operational analysis
5	Mixed-flow operational performance	Operational analysis
6	Severe weather effects on a basic freeway segment	Operational analysis
7	Basic managed lane segment	Operational analysis

EXAMPLE PROBLEM 1: FOUR-LANE FREEWAY LOS

The Facts

- Four-lane freeway (two lanes in each direction)
- Lane width = 11 ft
- Right-side lateral clearance = 2 ft
- Commuter traffic (regular users)
- Peak hour, peak direction demand volume = 2,000 veh/h
- Traffic composition: 5% trucks
- Peak hour factor (PHF) = 0.92
- One cloverleaf interchange per mile
- Level terrain
- Facility operates under ideal conditions (no incidents, work zones, or weather events).

Comments

The task is to find the expected LOS for this freeway during the worst 15 min of the peak hour. With one cloverleaf interchange per mile, the total ramp density will be 4 ramps/mi.

Step 1: Input Data

All input data are specified above.

Step 2: Estimate and Adjust FFS

The FFS of the freeway is estimated from Equation 12-2 as follows:

$$FFS = 75.4 - f_{LW} - f_{RLC} - 3.22 \times TRD^{0.84}$$

The adjustment for lane width is selected from Exhibit 12-20 for 11-ft lanes (1.9 mi/h). The adjustment for right-side lateral clearance is selected from Exhibit 12-21 for a 2-ft clearance on a freeway with two lanes in one direction (2.4 mi/h). The total ramp density is 4 ramps/mi. Then

$$FFS = 75.4 - 1.9 - 2.4 - 3.22 \times 4^{0.84} = 60.8 \text{ mi/h}$$

Because the facility is operating under ideal conditions, the SAF used in Equation 12-5 is 1, and $FFS_{adj} = FFS$.

Step 3: Estimate and Adjust Capacity

The capacity of the freeway is estimated from Equation 12-6 as follows:

$$c = 2{,}200 + 10 \times (FFS_{adj} - 50)$$

$$c = 2{,}200 + 10 \times (60.8 - 50) = 2{,}308 \text{ pc/h/ln}$$

Because the facility is operating under ideal conditions, the CAF used in Equation 12-8 is 1, and $c_{adj} = c$.

Step 4: Adjust Demand Volume

The demand volume must be adjusted to a flow rate that reflects passenger cars per hour per lane under equivalent base conditions by using Equation 12-9.

$$v_p = \frac{V}{PHF \times N \times f_{HV}}$$

The demand volume is given as 2,000 veh/h. The PHF is specified to be 0.92, and there are two lanes in each direction. The driver population consists of regular users (commuters). Trucks make up 5% of the traffic stream, so a heavy-vehicle adjustment factor must be determined.

From Exhibit 12-25, the PCE for trucks is 2.0 for level terrain. The heavy-vehicle adjustment factor is then computed with Equation 12-10.

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

$$f_{HV} = \frac{1}{1 + 0.05(2.0 - 1)} = 0.952$$

then

$$v_f = \frac{2{,}000}{0.92 \times 2 \times 0.952 \times 1.00} = 1{,}142 \text{ pc/h/ln}$$

Because this value is less than the base capacity of 2,308 pc/h/ln for a freeway with FFS = 60.8 mi/h, LOS F does not exist, and the analysis continues to Step 5.

Step 5: Estimate Speed and Density

The FFS of the basic freeway segment is now estimated along with the demand flow rate (in passenger cars per hour per lane) under equivalent base

conditions. Using the equations provided in Exhibit 12-6, the breakpoint for a 60.8-mi/h FFS speed–flow curve is

$$BP_{adj} = [1{,}000 + 40 \times (75 - FFS_{adj})] \times CAF^2 = 1{,}568 \text{ pc/h/ln}$$

As the flow rate of 1,142 pc/h/ln is less than the breakpoint value of 1,568 pc/h/ln, the freeway operates within the constant-speed portion of the speed–flow curve, so $S = 60.8$ mi/h. The density of the traffic stream may now be computed from Equation 12-11.

$$D = \frac{v_p}{S} = \frac{1{,}142}{60.8} = 18.8 \text{ pc/mi/ln}$$

Step 6: Determine LOS

From Exhibit 12-15, a density of 18.8 pc/mi/ln corresponds to LOS C, but it is close to the boundary for LOS B, which is a maximum of 18 pc/mi/ln. This solution could also be calculated graphically from Exhibit 12-16, as illustrated in Exhibit 26-16.

Exhibit 26-16
Example Problem 1: Graphical Solution
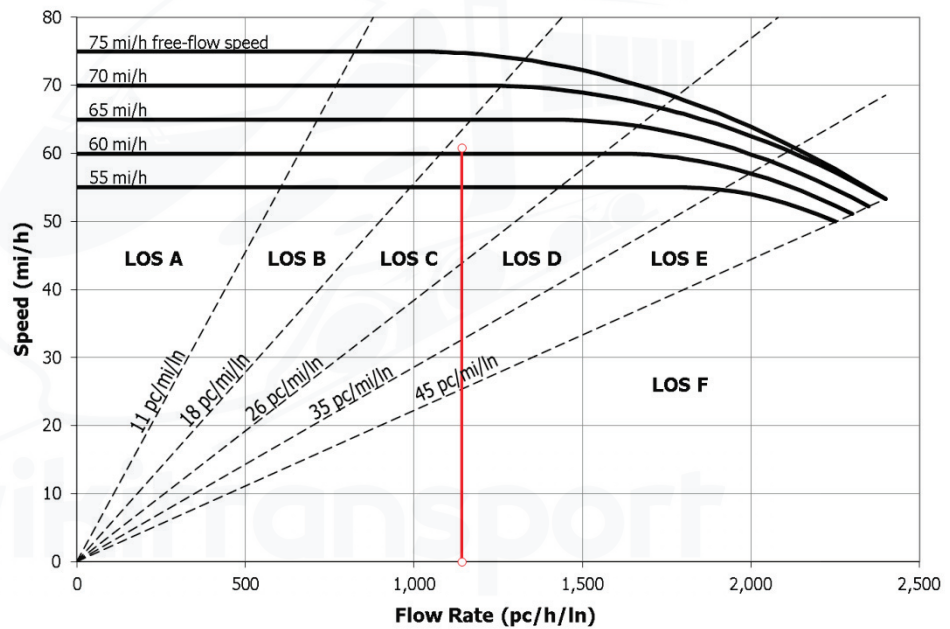
Discussion

This basic freeway segment of a four-lane freeway is expected to operate at LOS C during the worst 15 min of the peak hour. It is important to note that the operation, although at LOS C, is close to the LOS B boundary. In most jurisdictions, this operation would be considered to be quite acceptable.

EXAMPLE PROBLEM 2: NUMBER OF LANES REQUIRED FOR TARGET LOS

The Facts

- Demand volume = 4,000 veh/h (one direction)
- Level terrain
- Traffic composition: 8% SUTs and buses
- Provision of 12-ft lanes
- Provision of 6-ft right-side lateral clearance
- Commuter traffic (regular users)
- PHF = 0.85
- Ramp density = 3 ramps/mi
- Target LOS = D
- Facility operates under ideal conditions (no incidents, work zones, or weather events).

Comments

This example problem is a classic design application of the methodology. The number of lanes needed to provide LOS D during the worst 15 min of the peak hour is to be determined.

Step 1: Input Data

All input data are specified above.

Step 2: Estimate and Adjust FFS

FFS is estimated by using Equation 12-2. Because the lane width and lateral clearance to be provided on the new freeway will be 12 ft and 6 ft, respectively, there are no adjustments for these features from Exhibit 12-20 or Exhibit 12-21. The total ramp density is given as 3 ramps/mi. Then

$$FFS = 75.4 - f_{LW} - f_{RLC} - 3.22 \times TRD^{0.84}$$
$$FFS = 75.4 - 0 - 0 - 3.22 \times 3^{0.84} = 67.3 \text{ mi/h}$$

Because the facility is operating under ideal conditions, the SAF used in Equation 12-5 is 1, and $FFS_{adj} = FFS$.

Step 3: Estimate and Adjust Capacity

The capacity of the freeway is estimated from Equation 12-6.

$$c = 2,200 + 10 \times (FFS_{adj} - 50)$$
$$c = 2,200 + 10 \times (67.3 - 50) = 2,373 \text{ pc/h/ln}$$

Because the facility is operating under ideal conditions, the CAF used in Equation 12-8 is 1, and $c_{adj} = c$.

Step 4: Estimate Number of Lanes Needed

Because this is a design analysis, Step 4 of the operational analysis methodology is modified. Equation 12-23 may be used directly to determine the number of lanes needed to provide at least LOS D.

$$N = \frac{V}{MSF_i \times PHF \times f_{HV}}$$

A value of the maximum service flow rate must be selected from Exhibit 12-37 for an FFS of 65 mi/h and LOS D. Note that this exhibit only provides these values in 5-mi/h increments; therefore, FFS is rounded to 65 mi/h. The corresponding maximum service flow rate is 2,030 pc/h/ln.

The PHF is given as 0.85. A heavy-vehicle factor for 8% trucks must be determined by using Exhibit 12-25 for level terrain. The PCE of trucks on level terrain is 2.0, so the heavy-vehicle adjustment based on Equation 12-10 is

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

$$f_{HV} = \frac{1}{1 + 0.08(2 - 1)} = 0.926$$

and

$$N = \frac{4,000}{2,030 \times 0.85 \times 0.926 \times 1.00} = 2.5 \text{ ln}$$

It is not possible to build 2.5 lanes. To provide a minimum of LOS D, it will be necessary to provide three lanes in each direction, or a six-lane freeway.

At this point, the design application ends. It is possible, however, to consider what speed, density, and LOS will prevail when three lanes are actually provided. Therefore, the example problem continues with Steps 5 and 6.

Step 5: Estimate Speed and Density

In pursuing additional information, the problem now reverts to an operational analysis of a three-lane basic freeway segment with a demand volume of 4,000 pc/h.

Equation 12-9 is used to compute the actual demand flow rate per lane under equivalent base conditions.

$$v_p = \frac{V}{PHF \times N \times f_{HV}}$$

$$v_P = \frac{4,000}{0.85 \times 3 \times 0.926} = 1,694 \text{ pc/h/ln}$$

From Exhibit 12-6, the breakpoint for a speed–flow curve with FFS equal to 67.3 is

$$BP_{adj} = \left[1,000 + 40 \times \left(75 - FFS_{adj}\right)\right] \times CAF^2 = 1,308 \text{ pc/h/ln}$$

In this case, the demand flow rate of 1,694 pc/h/ln exceeds the breakpoint value of 1,308 pc/h/ln, and the average speed will be less than the FFS.

The expected speed of the traffic stream may be estimated by using either Exhibit 12-7 (for a graphical solution) or Equation 12-1 as follows:

$$S = FFS_{adj} - \frac{\left(FFS_{adj} - \frac{c_{adj}}{D_c}\right)\left(v_p - BP\right)^a}{\left(c_{adj} - BP\right)^a}$$

$$S = 67.3 - \frac{\left(67.3 - \frac{2{,}373}{45}\right)(1{,}694 - 1{,}308)^2}{(2{,}373 - 1{,}308)^2} = 65.4 \text{ mi/h}$$

The density may now be computed from Equation 12-11.

$$D = \frac{v_p}{S} = \frac{1{,}694}{65.4} = 25.9 \text{ pc/mi/ln}$$

Step 6: Determine LOS

Entering Exhibit 12-15 with a density of 25.9 pc/mi/ln, the LOS is C, but that density is very close to the boundary of LOS D, which is 26 pc/mi/ln.

Discussion

The resulting LOS is C, which represents a better performance than the target design. Although the minimum number of lanes needed was 2.5, which would have produced a minimal LOS D, providing three lanes yields a density that is close to the LOS C boundary. In any event, the target LOS of the design will be met by providing a six-lane basic freeway segment.

EXAMPLE PROBLEM 3: SIX-LANE FREEWAY LOS AND CAPACITY

The Facts

- Volume of 5,000 veh/h (one direction, existing)
- Volume of 5,788 veh/h (one direction, in 3 years)
- Traffic composition: 4% trucks
- Rolling terrain
- Three lanes in each direction
- FFS = 70 mi/h (measured)
- PHF = 0.96
- Commuter traffic (regular users)
- Traffic growth = 5% per year
- Facility operates under ideal conditions (no incidents, work zones, or weather events).

Comments

This example consists of two operational analyses, one for the present demand volume of 5,000 pc/h and one for the demand volume of 5,788 pc/h expected in 3 years. In addition, a planning element is introduced: Assuming traffic grows as expected, when will the capacity of the roadway be exceeded? This analysis requires that capacity be determined in addition to the normal output of operational analyses.

Step 1: Input Data

All input data are specified above.

Step 2: Estimate and Adjust FFS

Step 2 is not needed, as the FFS was directly measured (70 mi/h). Because the facility is operating under ideal conditions, the SAF used in Equation 12-5 is 1, and $FFS_{adj} = FFS$.

Step 3: Estimate and Adjust Capacity

The capacity of the freeway is estimated from Equation 12-6.

$$c = 2{,}200 + 10 \times (FFS_{adj} - 50)$$

$$c = 2{,}200 + 10 \times (70 - 50) = 2{,}400 \text{ pc/h/ln}$$

Because the facility is operating under ideal conditions, the CAF used in Equation 12-8 is 1, and $c_{adj} = c$.

Step 4: Adjust Demand Volume

In this case, two demand volumes will be adjusted by using Equation 12-9.

$$v_p = \frac{V}{PHF \times N \times f_{HV}}$$

The PHF is given as 0.96, and there are three lanes in each direction. The heavy-vehicle factor will reflect 4% trucks in rolling terrain. From Exhibit 12-25, the PCE for trucks in rolling terrain is 3.0. Equation 12-10 then gives

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

$$f_{HV} = \frac{1}{1 + 0.04(3.0 - 1)} = 0.926$$

Two values of v_p are computed: one for present conditions and one for conditions in 3 years.

$$v_p(\text{present}) = \frac{5{,}000}{0.96 \times 3 \times 0.926} = 1{,}875 \text{ pc/h}$$

$$v_p(\text{future}) = \frac{5{,}788}{0.96 \times 3 \times 0.926} = 2{,}171 \text{ pc/h}$$

Step 5: Estimate Speed and Density

Two values of speed and density will be estimated, one each for the present and future conditions. Equation 12-1 will be used to estimate speeds. First, the breakpoint for the speed–flow curve is computed from Exhibit 12-6.

$$BP_{adj} = \left[1{,}000 + 40 \times (75 - FFS_{adj})\right] \times CAF^2 = 1{,}200 \text{ pc/h/ln}$$

One equation applies to both cases; a 70-mi/h FFS with a flow rate over 1,200 pc/h/ln is used.

$$S = FFS_{adj} - \frac{\left(FFS_{adj} - \frac{c_{adj}}{D_c}\right)(v_p - BP)^a}{\left(c_{adj} - BP\right)^a}$$

$$S(\text{present}) = 70 - \frac{\left(70 - \frac{2{,}400}{45}\right)(1{,}875 - 1{,}200)^2}{(2{,}400 - 1{,}200)^2} = 64.7 \text{ mi/h}$$

$$S(\text{future}) = 70 - \frac{\left(70 - \frac{2{,}400}{45}\right)(2{,}171 - 1{,}200)^2}{(2{,}400 - 1{,}200)^2} = 59.1 \text{ mi/h}$$

The corresponding densities may now be estimated from Equation 12-11.

$$D = \frac{v_P}{S}$$

$$D(\text{present}) = \frac{1{,}875}{64.7} = 29.0 \text{ pc/mi/ln}$$

$$D(\text{future}) = \frac{2{,}171}{59.1} = 36.7 \text{ pc/mi/ln}$$

Step 6: Determine LOS

From Exhibit 12-15, the LOS for the present situation is D, and the LOS for the future scenario (in 3 years) is E due to the increase in density.

Step 7: Determine When Capacity Will Be Reached

Step 7 is an additional step for this problem. To determine when capacity will be reached, the capacity of the basic freeway segment must be estimated. From Exhibit 12-37, the maximum service flow rate for LOS E on a basic freeway segment with a 70-mi/h FFS is 2,400 pc/h/ln. This flow rate is synonymous with capacity.

The analyst must be sure the capacity and demand flow rates compared in Step 7 are measured on the same basis. The 2,400 pc/h/ln is a flow rate under equivalent base conditions. The demand flow rate in 3 years was estimated to be 2,171 pc/h/ln on this basis. These two values, therefore, may be compared. As an alternative, the capacity could be computed for prevailing conditions with Equation 12-24.

$$SF_E = MSF_E \times N \times f_{HV}$$
$$SF_E = 2{,}400 \times 3 \times 0.926 = 6{,}667 \text{ veh/h}$$

This capacity, however, is stated as a *flow rate*. The demand volume is stated as an hourly volume. Thus, a *service volume* for LOS E is needed as estimated from Equation 12-25.

$$SV_E = SF_E \times PHF = 6{,}667 \times 0.96 = 6{,}400 \text{ veh/h}$$

The problem may be solved either by comparing the demand volume of 5,788 veh/h (in 3 years) with the hourly capacity of 6,400 veh/h or by comparing the demand flow rate under equivalent base conditions of 2,171 pc/h/ln with the base capacity of 2,400 pc/h/ln. With the hourly demand volume and capacity,

$$6{,}400 = 5{,}788 \times (1.05)^n$$

$$n = 2.06 \text{ years}$$

On the basis of the forecasts of traffic growth, the basic freeway segment described will reach capacity within 5 years. The demand value of 5,788 veh/h occurs 3 years from the present per the problem description, and the calculation above shows capacity is reached after an additional 2 years. If this result is added to the 3-year planning horizon, capacity will be reached within 5 years of the time of the analysis.

Discussion

The LOS on this segment will reach LOS E within 3 years due to the increase in density. The demand is expected to exceed capacity within 5 years. Given the normal lead times for planning, design, and approvals before the start of construction, it is probable that planning and preliminary design for an improvement should be started immediately.

EXAMPLE PROBLEM 4: LOS ON A FIVE-LANE HIGHWAY WITH A TWO-WAY LEFT-TURN LANE

The Facts

- Lane width: 12 ft
- Lateral clearance, both sides of the roadway: 12 ft
- Traffic composition: 6% trucks, with default truck mix (30% SUTs, 70% TTs)
- Access points per mile: eastbound = 10; westbound = 0
- PHF = 0.90
- Commuter traffic (regular users)
- Median type: two-way left-turn lane
- Peak hour demand: 1,500 veh/h
- The upgrade occurs in the westbound direction
- Posted speed limit = 45 mi/h

Comments

A 6,600-ft segment of a five-lane highway (two travel lanes in each direction plus a two-way left-turn lane) is on a 3.5% grade. At what LOS is the facility expected to operate in each direction?

There is one segment in each direction. The upgrade and downgrade segments on the 3.5% grade must be analyzed separately. This example is more complex than the previous examples because the segment characteristics are not all the same, particularly the number of access points. Because no base FFS is given, it will be estimated as the speed limit plus 7 mi/h, or 45 + 7 = 52 mi/h.

Step 1: Input Data

All input data are given above.

Step 2: Estimate and Adjust FFS

FFS is estimated by using Equation 12-3.

$$FFS = BFFS - f_{LW} - f_{TLC} - f_M - f_A$$

In this case, the base FFS is estimated to be 52 mi/h. The lane width is 12 ft, which is the base condition; therefore, f_{LW} = 0.0 mi/h (Exhibit 12-20). The lateral clearance is 12 ft at each roadside, but a maximum value of 6 ft may be used. A two-way left-turn lane is considered to have a median lateral clearance of 6 ft. Thus, the total lateral clearance is 6 + 6 = 12 ft, which is also a base condition.

Therefore, $f_{TLC} = 0.0$ mi/h (Exhibit 12-22). The median-type adjustment f_M is also 0.0 mi/h (Exhibit 12-23).

For this example problem, only the access-point density produces a nonzero adjustment to the base FFS. The eastbound (EB) segment (3.5% downgrade) has 10 access points/mi. From Exhibit 12-24, the corresponding FFS adjustment is 2.5 mi/h. The westbound (WB) segment (3.5% upgrade) has 0 access points/mi and a corresponding FFS adjustment of 0.0 mi/h. Therefore,

$$FFS_{EB} = 52.0 - 0.0 - 0.0 - 0.0 - 2.5 = 49.5 \text{ mi/h}$$
$$FFS_{WB} = 52.0 - 0.0 - 0.0 - 0.0 - 0.0 = 52.0 \text{ mi/h}$$

Step 3: Estimate and Adjust Capacity

The capacity of the multilane highway segment is estimated as follows from Equation 12-7.

$$c = 1,900 + 20 \times (FFS_{adj} - 45)$$
$$c_{EB} = 1,900 + 20 \times (49.5 - 45) = 1,990 \text{ pc/h/ln}$$
$$c_{WB} = 1,900 + 20 \times (52.0 - 45) = 2,040 \text{ pc/h/ln}$$

Step 4: Adjust Demand Volume

Demand volume is adjusted by using Equation 12-9.

$$v_p = \frac{V}{PHF \times N \times f_{HV}}$$

To compute the heavy-vehicle adjustment factor f_{HV}, PCEs for trucks are needed for (*a*) the 3.5%, 6,600-ft upgrade and (*b*) the 3.5%, 6,600-ft downgrade. The segment is 1.25 mi (6,600/5,280 ft) long. The following values are obtained from Exhibit 12-26:

- Eastbound: 2.24 (using 6% trucks, a 2% downgrade, and 1.25-mi grade length). Note that all downgrades exceeding 2% use the PCE values for a 2% downgrade.

- Westbound: 3.97 (using 6% trucks, a 3.5% upgrade, and a 1.25-mi grade length).

The heavy-vehicle adjustment factors f_{HV} for each segment are calculated from Equation 12-10.

$$f_{HV,EB} = \frac{1}{1 + 0.06 \times (2.24 - 1)} = 0.93$$
$$f_{HV,WB} = \frac{1}{1 + 0.06 \times (3.97 - 1)} = 0.85$$

The segments' flow rates are then calculated as

$$v_{p,EB} = \frac{1,500}{0.90 \times 2 \times 0.93} = 896 \text{ pc/h/ln}$$
$$v_{p,WB} = \frac{1,500}{0.90 \times 2 \times 0.85} = 980 \text{ pc/h/ln}$$

Step 5: Estimate Speed and Density

Speed is estimated with Equation 12-1 or the graph in Exhibit 12-7. With Equation 12-1, both demand flow rates are less than the multilane highway breakpoint value of 1,400 pc/h/ln. Therefore, the speeds *S* are equal to FFS. The densities are computed from Equation 12-11.

$$D_{EB} = \frac{v_{p,EB}}{S_{EB}} = \frac{896}{49.5} = 18.1 \text{ pc/mi/ln}$$

$$D_{WB} = \frac{v_{p,WB}}{S_{WB}} = \frac{980}{52} = 18.8 \text{ pc/mi/ln}$$

Step 6: Determine LOS

LOS is found by comparing the densities of the segments with the criteria in Exhibit 12-15. As both densities are greater than 18 pc/mi/ln, both upgrade and downgrade segments operate at LOS C.

Discussion

Even though the upgrade and downgrade segments operate at LOS C, they are very close to the LOS B boundary (18.0 pc/mi/ln). Both directions of the multilane highway on this grade operate well.

EXAMPLE PROBLEM 5: MIXED-FLOW FREEWAY OPERATIONS

This example illustrates the application of the mixed-flow model for an extended single grade on a six-lane rural freeway.

The Facts

- 2-mi basic segment on a 5% upgrade
- Traffic composition: 5% SUTs and 10% TTs
- FFS = 65 mi/h
- Mixed-traffic flow rate = 1,500 veh/h/ln

Comments

The task is to estimate the segment's speed and density. Given the significant truck presence (15%) and the 5%, 2-mi grade, the mixed-flow model should be applied.

Step 1: Input Data

All input data are specified above.

Step 2: Compute Mixed-Flow Capacity Adjustment Factor

Capacity is computed with Equation 26-1.

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

There are three terms in the equation. The CAF for auto-only CAF_{ao} is 1.00, as no driver population, weather, incident, or work zone adjustments are specified in the problem statement.

The truck effect term is computed with Equation 26-2.

$$CAF_{T,\text{mix}} = 0.53 \times P_T^{0.72} = 0.53 \times 0.15^{0.72} = 0.135$$

The grade effect term is computed with Equation 26-3 and Equation 26-4.

$$CAF_{g,\text{mix}} = \rho_{g,\text{mix}} \times \max[0, 0.69 \times (e^{12.9g} - 1)]$$
$$\times \max[0, 1.72 \times (1 - 1.71e^{-3.16d})]$$

$$\rho_{g,\text{mix}} = 0.126 - 0.03P_T = 0.126 - (0.03)(0.15) = 0.1215$$

$$CAF_{g,\text{mix}} = 0.1215 \times \max\left[0, 0.69 \times \left(e^{(12.9)(0.05)} - 1\right)\right]$$
$$\times \max\left[0, 1.72 \times \left(1 - 1.71e^{(-3.16)(2)}\right)\right] = 0.131$$

then

$$CAF_{\text{mix}} = 1 - 0.135 - 0.131 = 0.734$$

The mixed-flow capacity is then computed from Equation 26-5.

$$C_{\text{mix}} = C_{ao} \times CAF_{\text{mix}}$$

The auto-only capacity C_{ao} is computed from Exhibit 12-6.

$$C_{ao} = 2,200 + 10(FFS - 50) = 2,200 + 10 \times (65 - 50) = 2,350 \text{ pc/h/ln}$$

then

$$C_{\text{mix}} = 2,350 \times 0.734 = 1,725 \text{ veh/h/ln}$$

As the mixed-traffic flow rate of 1,500 veh/h/ln is less than the mixed-flow capacity of 1,725 veh/h/ln, the analysis can proceed.

Step 3: Compute Mixed-Flow FFS and FFS Adjustment Factor

Equation 26-6 through Equation 26-8 compute the free-flow travel rates for SUTs, TTs, and automobiles, respectively. The FFS of this basic freeway segment is 65 mi/h. Truck performance curves for free-flow speeds other than 70 ± 2.5 mi/h are provided in Appendix A. The 65-mi/h curves for SUTs and TTs are found in Exhibit 26-A4 and Exhibit 26-A9, respectively.

The travel time for a SUT T_{SUT} at a point 10,000 ft along the upgrade can be read directly from Exhibit 26-A4 by observing where the 5% upgrade curve intersects 10,000 ft: 134 s. Similarly, the travel time for a TT T_{TT} is 173 s.

As the grade is 2 mi (10,560 ft) long and the performance curves only provide values up to 10,000 ft, Equation 26-12 is used to determine the travel time rates for the upgrade as a whole. The slope of the travel time versus distance curve δ, which is used in Equation 26-12, can be determined from Exhibit 26-7 for SUTs and Exhibit 26-8 for TTs. The δ values are 0.0146 and 0.0202, respectively.

Then

$$\tau_{kin,SUT} = \frac{T_{SUT,10000ft}}{d} + \delta\left(1 - \frac{10,000}{5280d}\right) \times 5,280$$

$$\tau_{kin,SUT} = \frac{134}{2} + 0.0146\left(1 - \frac{10,000}{10,560}\right) \times 5,280 = 71.1 \text{ s/mi}$$

$$\tau_{kin,TT} = \frac{173}{2} + 0.0202\left(1 - \frac{10,000}{10,560}\right) \times 5,280 = 92.2 \text{ s/mi}$$

As this step's objective is to compute the FFS of the mixed-traffic stream, the traffic interaction term $\Delta\tau_{TI}$ is zero, and the mixed-flow rate is set to 1 veh/h/ln.

The SUT, TT, and auto travel time rates are then computed using Equation 26-6 through Equation 26-8.

$$\tau_{SUT,FFS} = 71.1 + 0 = 71.1 \text{ s/mi}$$

$$\tau_{TT,FFS} = 92.2 + 0 = 92.2 \text{ s/mi}$$

$$\tau_{a,FFS} = \frac{3,600}{FFS} + \Delta\tau_{TI}$$

$$+ 100.42 \times \left(\frac{v_{\text{mix}}}{1,000}\right)^{0.46} \times P_{SUT}{}^{0.68} \times \max\left[0, \frac{\tau_{kin,SUT}}{100} - \frac{3,600}{(FFS \times 100)}\right]^{2.76}$$

$$+ 110.64 \times \left(\frac{v_{\text{mix}}}{1,000}\right)^{1.36} \times P_{TT}{}^{0.62} \times \max\left[0, \frac{\tau_{kin,TT}}{100} - \frac{3,600}{(FFS \times 100)}\right]^{1.81}$$

$$\tau_{a,FFS} = \frac{3,600}{65} + 0$$

$$+ 100.42 \times \left(\frac{1}{1,000}\right)^{0.46} \times 0.05^{0.68} \times \max\left[0, \frac{71.1}{100} - \frac{3,600}{(65 \times 100)}\right]^{2.76}$$

$$+ 110.64 \times \left(\frac{1}{1,000}\right)^{1.36} \times 0.1^{0.62} \times \max\left[0, \frac{92.2}{100} - \frac{3,600}{(65 \times 100)}\right]^{1.81}$$

$$\tau_{a,FFS} = 55.4 \text{ s/mi}$$

Mixed-flow travel rates and speeds are computed with Equation 26-13 and Equation 26-14.

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

$$\tau_{\text{mix},FFS} = (0.85)(55.4) + (0.05)(71.1) + (0.1)(92.2) = 59.87 \text{ s/mi}$$

$$FFS_{\text{mix}} = \frac{3,600}{\tau_{\text{mix},FFS}} = \frac{3,600}{59.87} = 60.1 \text{ mi/h}$$

Finally, the segment's SAF is estimated with Equation 26-15.

$$SAF_{\text{mix}} = \frac{FFS_{\text{mix}}}{FFS} = \frac{60.1}{65} = 0.92$$

Step 4: Compute the Mixed-Flow Rate at the Breakpoint

The breakpoint is calculated from Equation 26-16.

$$BP_{\text{mix}} = \max\left[0, BP_{ao}(1 - 0.4P_T{}^{0.1} \times \max[0, e^{30g} + 1] \times d^{0.01})\right]$$

where the auto-only breakpoint is calculated by using an equation given in Exhibit 12-6.

$$BP_{ao} = [1,000 + 40 \times (75 - FFS)] \times CAF^2$$

$$BP_{ao} = [1,000 + 40 \times (75 - 65)] \times 1^2 = 1,400 \text{ veh/h/ln}$$

then

$$BP_{\text{mix}} = \max[0, (1,400)(1 - 0.4(0.15)^{0.1} \times \max[0, e^{30\times0.05} + 1] \times 2^{0.01})]$$

$$BP_{\text{mix}} = 0 \text{ veh/h/ln}$$

This result implies that speeds drop immediately at zero flow (i.e., the mixed-flow FFS cannot be sustained even at low flows).

Step 5: Compute Modal and Mixed-Flow Speeds at Capacity and 90% of Capacity

The speeds and densities for each mode at capacity and 90% of capacity are calculated in this step. Equation 26-6 through Equation 26-8 are applied twice more, once for a flow rate equal to the mixed-flow capacity of 1,725 veh/h/ln calculated in Step 2, and again for a flow rate equal to 90% of capacity. Applying these equations requires determining the traffic interaction term $\Delta\tau_{TI}$, which in turn requires determining the equivalent auto-only speed S_{ao}.

The calculation process will be demonstrated for conditions at capacity. The value of C_{mix} determined in Step 2 (1,725 veh/h/ln) will be used as v_{mix} in the calculations.

The auto-only speed at capacity is computed by Equation 26-10.

$$S_{ao} = \begin{cases} FFS & \dfrac{v_{mix}}{CAF_{mix}} \leq BP_{ao} \\[2em] FFS - \dfrac{\left(FFS - \dfrac{c}{D_c}\right)\left(\dfrac{v_{mix}}{CAF_{mix}} - BP_{ao}\right)^2}{(c - BP_{ao})^2} & \dfrac{v_{mix}}{CAF_{mix}} > BP_{ao} \end{cases}$$

The value of v_{mix}/CAF_{mix} is 1,725/0.734 = 2,350 veh/h/ln, which is greater than the auto-only breakpoint of 1,400 veh/h/ln calculated in Step 4. Therefore, the second of the two equations is applied.

$$S_{ao,cap} = 65 - \frac{\left(65 - \dfrac{2,350}{45}\right)\left(\dfrac{1,725}{0.734} - 1,400\right)^2}{(2,350 - 1,400)^2} = 52.2 \text{ mi/h}$$

The traffic interaction term can now be computed with Equation 26-9.

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

$$\Delta\tau_{TI,cap} = \left(\frac{3,600}{52.2} - \frac{3,600}{65}\right) \times \left(1 + 3\left[\frac{1}{0.734} - 1\right]\right) = 28.3 \text{ s/mi}$$

Equation 26-6 through Equation 26-8 are now applied to find the modal travel time rates at capacity.

$$\tau_{SUT,cap} = \tau_{SUT,kin} + \Delta\tau_{TI} = 71.1 + 28.3 = 99.4 \text{ s/mi}$$

$$\tau_{TT,cap} = \tau_{TT,kin} + \Delta\tau_{TI} = 92.2 + 28.3 = 120.5 \text{ s/mi}$$

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

$$\tau_{a,cap} = \frac{3,600}{65} + 28.3$$

$$+100.42 \times \left(\frac{1,725}{1,000}\right)^{0.46} \times 0.05^{0.68} \times \max\left[0, \frac{71.1}{100} - \frac{3,600}{(65 \times 100)}\right]^{2.76}$$

$$+110.64 \times \left(\frac{1,725}{1,000}\right)^{1.36} \times 0.1^{0.62} \times \max\left[0, \frac{92.2}{100} - \frac{3,600}{(65 \times 100)}\right]^{1.81}$$

$$\tau_{a,cap} = 92.9 \text{ s/mi}$$

Based on these travel rates, the overall mixed-traffic space mean speed at capacity can be calculated with Equation 26-19.

$$S_{calib,cap} = \frac{3,600}{P_a\tau_a + P_{SUT}\tau_{SUT} + P_{TT}\tau_{TT}}$$

$$S_{calib,cap} = \frac{3,600}{(0.85)(92.9) + (0.05)(99.4) + (0.1)(120.5)} = 37.5 \text{ mi/h}$$

The same process is used to calculate the mixed-traffic speed at 90% of capacity (v_{mix} = 0.9 × 1,725 = 1,553 veh/h/ln). The resulting calculation results are

$$S_{ao,90cap} = 65 - \frac{\left(65 - \frac{2,350}{45}\right)\left(\frac{1,553}{0.734} - 1,400\right)^2}{(2,350 - 1,400)^2} = 57.7 \text{ mi/h}$$

$$\Delta\tau_{TI,90cap} = \left(\frac{3,600}{57.7} - \frac{3,600}{65}\right) \times \left(1 + 3\left[\frac{1}{0.734} - 1\right]\right) = 14.6 \text{ s/mi}$$

$$\tau_{SUT,90cap} = 71.1 + 14.6 = 85.7 \text{ s/mi}$$

$$\tau_{TT,90cap} = 92.2 + 14.6 = 106.8 \text{ s/mi}$$

$$\tau_{a,90cap} = \frac{3,600}{65} + 14.6$$

$$+100.42 \times \left(\frac{1,553}{1,000}\right)^{0.46} \times 0.05^{0.68} \times \max\left[0, \frac{71.1}{100} - \frac{3,600}{(65 \times 100)}\right]^{2.76}$$

$$+110.64 \times \left(\frac{1,553}{1,000}\right)^{1.36} \times 0.1^{0.62} \times \max\left[0, \frac{92.2}{100} - \frac{3,600}{(65 \times 100)}\right]^{1.81}$$

$$\tau_{a,90cap} = 78.0 \text{ s/mi}$$

$$S_{calib,90cap} = \frac{3,600}{(0.85)(78.0) + (0.05)(85.7) + (0.1)(106.8)} = 44.3 \text{ mi/h}$$

Step 6: Compute the Exponent for the Speed–Flow Curve

The exponent for the speed–flow curve is computed from Equation 26-20.

$$\phi_{mix} = 1.195 \times \frac{\ln\left(\frac{FFS_{mix} - S_{calib,90cap}}{FFS_{mix} - S_{calib,cap}}\right)}{\ln\left(\frac{0.9C_{mix} - BP_{mix}}{C_{mix} - BP_{mix}}\right)}$$

$$\phi_{mix} = 1.195 \times \frac{\ln\left(\frac{60.1 - 44.3}{60.1 - 37.5}\right)}{\ln\left(\frac{1,553 - 0}{1,725 - 0}\right)} = 4.07$$

Step 7: Compute the Mixed-Flow Speed Under Mixed-Flow Conditions

The mixed-flow speed under mixed-flow conditions is computed by Equation 26-21.

$$
S_{\text{mix}} = \begin{cases} FFS_{\text{mix}} & v_{\text{mix}} \leq BP_{\text{mix}} \\ FFS_{\text{mix}} - \left(FFS_{\text{mix}} - S_{calib,cap}\right)\left(\dfrac{v_{\text{mix}} - BP_{\text{mix}}}{C_{\text{mix}} - BP_{\text{mix}}}\right)^{\phi_{\text{mix}}} & v_{\text{mix}} > BP_{\text{mix}} \end{cases}
$$

The mixed-flow rate is 1,500 veh/h/ln, which is greater than the breakpoint. Therefore,

$$
S_{\text{mix}} = 60.1 - (60.1 - 37.5)\left(\frac{1,500 - 0}{1,725 - 0}\right)^{4.07} = 47.3 \text{ mi/h}
$$

Step 8: Compute the Mixed-Flow Density Under Mixed-Flow Conditions

The final step is to compute the mixed-flow density by using Equation 26-22.

$$
D_{\text{mix}} = \frac{v_{\text{mix}}}{S_{\text{mix}}} = \frac{1,500}{47.3} = 31.7 \text{ veh/mi/ln}
$$

Comparison with the PCE-Based Approach

For comparison purposes, the following procedure show the results for this case if the PCE-based approach explained in Chapter 12 is applied.

Step 1: Input Data

All input data are specified above.

Step 2: Estimate and Adjust FFS

For basic freeway segments, Equation 12-2 can be used to estimate FFS.

$$
FFS = BFFS - f_{LW} - f_{RLC} - 3.22 \times TRD^{0.84}
$$

For the purposes of comparing the two methods with respect to truck effects on FFS, the lane width, lateral clearance, and ramp density adjustment factors can be neglected. Then,

$$
FFS = 65 - 0 - 0 - 3.22 \times 0^{0.84} = 65 \text{ mi/h}
$$

The adjusted FFS is computed from Equation 12-5, assuming no weather or incident effects.

$$
FFS_{adj} = FFS \times SAF
$$
$$
FFS_{adj} = 65 \times 1 = 65 \text{ mi/h}
$$

Step 3: Estimate and Adjust Capacity

Equation 12-6 is used to compute the capacity of a basic freeway segment.

$$
c = 2,200 + 10 \times (FFS_{adj} - 50)
$$
$$
c = 2,200 + 10 \times (65 - 50) = 2,350 \text{ pc/h/ln}
$$

Assuming no adverse weather conditions or incidents, the adjusted capacity from Equation 12-8 is then

$$
c_{adj} = c \times CAF = 2,350 \times 1 = 2,350 \text{ pc/h/ln}
$$

Step 4: Adjust Demand Volume

This basic freeway segment is in a rural area with more TTs than SUTs. Therefore, the PCE table for 30% SUTs and 70% TTs (Exhibit 12-26) will be used. As stated in the Facts section of the example problem, the grade is 5% for 2 mi. There are no values specifically for a 5% grade in Exhibit 12-26; therefore, PCE values will be interpolated from the values for 4.5% and 5.5%. As the maximum grade length provided in the exhibit is 1 mi for these two grades, values for a 1-mi grade will also apply to longer grades. For a 1-mi, 4.5% grade, the PCE value for 15% trucks is 3.11; and the PCE value for a 1-mi, 5.5% grade with 15% trucks is 3.51. Interpolating between these two values for a 5% grade results in a PCE of 3.31.

The heavy-vehicle factor can be computed with Equation 12-10.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.15 \times (3.31 - 1)} = 0.743$$

Equation 12-9 is used to adjust the demand volume to account for truck presence. The freeway is a three-lane facility and the driver population is assumed to be all local drivers.

$$v_p = \frac{V}{PHF \times N \times f_{HV}} = \frac{1,500 \times 3}{1 \times 3 \times 0.743} = 2,019 \text{ pc/h/ln}$$

Step 5: Estimate Speed and Density

The speed can be read directly from Exhibit 12-7 for a demand flow rate of 2,019 pc/h/ln. Under base conditions, the mean speed of the traffic stream is 59.6 mi/h as calculated from Equation 26-1.

Equation 12-11 is used to compute density.

$$D = \frac{v_P}{S} = \frac{2,019}{59.6} = 33.9 \text{ pc/mi/ln}$$

If the density above is multiplied by the heavy-vehicle factor, then the mixed-flow density D_{mix} can be estimated as follows:

$$D_{mix} = D \times f_{HV} = 33.9 \times 0.743 = 25.2 \text{ veh/mi/ln}$$

The PCE-based density of 25.2 veh/mi/ln is about 22% lower than 32.6 veh/mi/ln, which is the density predicted in Step 8 of the mixed-flow model. D_{mix} is the mixed-flow density, not an auto-only flow density. As such, it cannot be used to derive LOS.

EXAMPLE PROBLEM 6: SEVERE WEATHER EFFECTS ON A BASIC FREEWAY SEGMENT

The Facts

- Four-lane freeway (two lanes in each direction)
- Lane width = 11 ft
- Right-side lateral clearance = 2 ft
- Commuter traffic (regular users)
- Peak hour, peak direction demand volume = 2,000 veh/h
- Traffic composition: 5% trucks
- PHF = 0.92
- One cloverleaf interchange per mile
- Rolling terrain
- Facility operates under heavy snow conditions (CAF = 0.78; SAF = 0.86).

Comments

The task is to find the expected LOS for this freeway during the worst 15 min of the peak hour under heavy snow conditions. With one cloverleaf interchange per mile, the total ramp density will be 4 ramps/mi. This example problem is similar to Example Problem 1, with the only change being the presence of heavy snow.

Step 1: Input Data

All input data are specified above.

Step 2: Estimate and Adjust FFS

The FFS of the freeway is estimated from Equation 12-2 as follows:

$$FFS = 75.4 - f_{LW} - f_{RLC} - 3.22 \times TRD^{0.84}$$

The adjustment for lane width is selected from Exhibit 12-20 for 11-ft lanes (1.9 mi/h). The adjustment for right-side lateral clearance is selected from Exhibit 12-21 for a 2-ft clearance on a freeway with two lanes in one direction (2.4 mi/h). The total ramp density is 4 ramps/mi. Then

$$FFS = 75.4 - 1.9 - 2.4 - 3.22 \times 4^{0.84} = 60.8 \text{ mi/h}$$

A free-flow speed adjustment factor (SAF) for heavy snow conditions can be obtained from Exhibit 11-5 in Chapter 11, Freeway Reliability Analysis, by interpolating between the values for 60 and 65 mi/h (0.86 and 0.85, respectively), resulting in a SAF of 0.86. No other speed adjustments are made, as no incidents were specified in the problem statement and because the driver population was specified to be commuters. The SAF is applied through Equation 12-5.

$$FFS_{adj} = FFS \times SAF = 60.8 \times 0.86 = 52.3 \text{ mi/h}$$

Step 3: Estimate and Adjust Capacity

Exhibit 11-5 also provides a CAF of 0.78 for heavy snow conditions, applicable to all FFS values. As with the SAF in Step 2, no other capacity adjustments apply in this situation. The freeway's capacity is then estimated using Equation 12-6.

$$c = CAF(2,200 + 10 \times [FFS_{adj} - 50])$$
$$c = 0.78 \times (2,200 + 10 \times [52.3 - 50]) = 1,734 \text{ pc/h/ln}$$

Step 4: Adjust Demand Volume

The demand volume is adjusted by using Equation 12-9 to a flow rate that reflects passenger cars per hour per lane under equivalent base conditions.

$$v_p = \frac{V}{PHF \times N \times f_{HV}}$$

The demand volume is given as 2,000 veh/h. The PHF is specified to be 0.92, and there are two lanes in each direction. Trucks make up 5% of the traffic stream, so a heavy-vehicle adjustment factor must be determined.

From Exhibit 12-25, the PCE for trucks is 3.0 for rolling terrain. The heavy-vehicle adjustment factor is then computed by using Equation 12-10.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.05(3 - 1)} = 0.909$$

then

$$v_p = \frac{2,000}{0.92 \times 2 \times 0.91} = 1,195 \text{ pc/h/ln}$$

Because this value is less than the base capacity of 1,743 pc/h/ln for a freeway with an FFS of 52.3 mi/h, LOS F conditions do not exist, and the analysis continues to Step 5.

Step 5: Estimate Speed and Density

The FFS of the basic freeway segment is now estimated along with the demand flow rate (in passenger cars per hour per lane) under equivalent base conditions. Using the equations provided in Exhibit 12-6, the breakpoint for a 53.5-mi/h FFS speed–flow curve is

$$BP_{adj} = [1,000 + 40 \times (75 - FFS_{adj})] \times (CAF)^2$$
$$BP_{adj} = [1,000 + 40 \times (75 - 52.3)] \times (0.78)^2 = 1,161 \text{ pc/h/ln}$$

Because the flow rate is greater than the breakpoint value, the operating speed of the segment is estimated from Equation 12-1, by using a value of 2 for the exponent calibration parameter *a* from Exhibit 12-6.

$$S = FFS_{adj} - \frac{\left(FFS_{adj} - \frac{c_{adj}}{D_c}\right)(v_p - BP)^a}{\left(c_{adj} - BP\right)^a}$$

$$S = 52.3 - \frac{\left(52.3 - \frac{1,734}{45}\right)(1,195 - 1,161)^2}{(1,734 - 1,161)^2} = 52.3 \text{ mi/h}$$

The density may now be computed from Equation 12-11.

$$D = \frac{v_p}{S} = \frac{1,195}{52.3} = 22.8 \text{ pc/mi/ln}$$

Step 6: Determine LOS

From Exhibit 12-15, a density of 22.8 pc/mi/ln corresponds to LOS C.

Discussion

This basic freeway segment of a four-lane freeway is expected to operate at LOS C during the worst 15 min of the peak hour under heavy snow conditions, with an average speed of 52.3 mi/h and a density of 22.8 pc/mi/ln. By contrast, the same facility under no adverse weather conditions would be expected to operate at an FFS of 60.8 mi/h and a density of 19.7 pc/mi/ln, but still at LOS C. Although the segment's performance is affected by the snow, the overall LOS is unchanged.

However, the segment's capacity is reduced from 2,308 to 1,734 pc/h/ln, which means the snow effect would be more severe at elevated volume-to-capacity ratios, particularly as the segment approached capacity. For elevated flow rates, the snow condition is expected to result in further deterioration of speed and breakdown at lower flow rates.

EXAMPLE PROBLEM 7: BASIC MANAGED LANE SEGMENT

The Facts

- Six-lane freeway with two general purpose lanes and one managed lane in each direction
- Lane width = 11 ft
- Right-side lateral clearance = 2 ft
- Commuter traffic (regular users)
- Peak hour, peak direction demand volume in the general purpose lanes = 2,000 veh/h (Case 1) or 3,800 veh/h (Case 2)
- Peak hour, peak direction demand volume in the managed lane (both cases) = 1,300 veh/h
- Continuous access separation between the managed and general purpose lanes
- FFS = 60 mi/h for both the managed and general purpose lanes
- Traffic composition: 7.5% trucks, using the default truck mix for both the managed and general purpose lanes
- PHF = 0.92
- One cloverleaf interchange per mile
- Level terrain
- Facility operates under ideal conditions (no incidents, work zones, or weather events).

Comments

The task is to find the expected LOS for this freeway for both the managed and general purpose lanes during the worst 15 min of the peak hour for the two described cases. With one cloverleaf interchange per mile, the total ramp density will be 4 ramps/mi.

Step 1: Input Data

All input data are specified above.

Step 2: Estimate and Adjust FFS

The facility's FFS is given as 60 mi/h for both the managed and general purpose lanes. Because the facility is operating under ideal conditions, the SAF used in Equation 12-5 is 1.

Step 3: Estimate and Adjust Capacity

The capacity of the freeway general purpose lanes is estimated from Equation 12-6 as follows:

$$c = 2{,}200 + 10 \times (FFS_{adj} - 50)$$
$$c = 2{,}200 + 10 \times (60 - 50) = 2{,}300 \text{ pc/h/ln}$$

As the freeway is operating under ideal conditions, no capacity adjustment is made for the general purpose lanes (i.e., CAF = 1 in Equation 12-8).

The capacity of the managed lane is calculated with Equation 12-14.

$$c_{adj} = CAF \times \left(c_{75} - \lambda_c \times [75 - FFS_{adj}]\right)$$

As with the general purpose lanes, CAF = 1 for the managed lane. The values of the parameters C_{75} and λ_c are obtained from Exhibit 12-30, and are 1,800 and 10, respectively, for continuous access separation. Then

$$c_{adj} = 1.00 \times (1{,}800 - 10 \times [75 - 60]) = 1{,}650 \text{ pc/h/ln}$$

Step 4: Adjust Demand Volume

The demand volume is adjusted by using Equation 12-9 to a flow rate that reflects passenger cars per hour per lane under equivalent base conditions.

$$v_p = \frac{V}{PHF \times N \times f_{HV}}$$

The demand volume is given as 2,000 veh/h and 3,800 veh/h for Cases 1 and 2, respectively. The PHF is specified to be 0.92, and there are two lanes in each direction. Trucks make up 5% of the traffic stream, so a heavy-vehicle adjustment factor must be determined.

From Exhibit 12-25, the PCE for trucks is 2.0 for level terrain. The heavy-vehicle adjustment factor is then computed using Equation 12-10.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)}\frac{1}{1 + 0.075(2.0 - 1)} = 0.93$$

Then for Case 1,

$$v_{p,GP,\text{Case1}} = \frac{2{,}000}{0.92 \times 2 \times 0.93} = 1{,}169 \text{ pc/h/ln}$$

and for Case 2,

$$v_{p,GP,\text{Case2}} = \frac{3,800}{0.92 \times 2 \times 0.93} = 2,221 \text{ pc/h/ln}$$

The flow rate on the managed lane is

$$v_{p,ML} = \frac{1,300}{0.92 \times 1 \times 0.93} = 1,519 \text{ pc/h/ln}$$

Because all the flow rates are less than their corresponding capacities, LOS F conditions do not exist, and the analysis continues to Step 5.

Step 5: Estimate Speed and Density

The FFS of the basic freeway segment is now estimated, along with the demand flow rate (in passenger cars per hour per lane) under equivalent base conditions. Based on the equations provided in Exhibit 12-6, the breakpoint for a 60-mi/h FFS speed–flow curve is

$$BP_{adj} = \left[1,000 + 40 \times (75 - FFS_{adj})\right] \times (CAF)^2$$
$$BP_{adj} = [1,000 + 40 \times (75 - 60)] \times (1.00)^2 = 1,600 \text{ pc/h/ln}$$

In Case 1, the flow rate is less than the breakpoint value of 1,600 pc/h/ln. As this flow rate is in the constant-speed portion of the curve, $S_{GP,\text{Case1}} = 60$ mi/h. The density of the traffic stream is computed from Equation 12-11.

$$D_{GP,\text{Case1}} = \frac{v_p}{S} = \frac{1,169}{60} = 19.5 \text{ pc/mi/ln}$$

In Case 2, the flow rate is higher than the breakpoint. Therefore, the speed is computed with Equation 12-1, by using a value of 2 for the exponent calibration parameter a from Exhibit 12-6, as follows:

$$S = FFS_{adj} - \frac{\left(FFS_{adj} - \frac{c_{adj}}{D_c}\right)\left(v_p - BP\right)^a}{\left(c_{adj} - BP\right)^a}$$

$$S_{GP,\text{Case2}} = 60 - \frac{\left(60 - \frac{2,300}{45}\right)(2,221 - 1,600)^2}{(2,300 - 1,600)^2} = 53.0 \text{ mi/h}$$

Density is computed with Equation 12-11.

$$D_{GP,\text{Case2}} = \frac{v_p}{S} = \frac{2,221}{53} = 41.9 \text{ pc/mi/ln}$$

To compute the managed lane speed, the breakpoint first needs to be computed by using Equation 12-13 and values for the parameters BP_{75} and λ_{BP} from Exhibit 12-30.

$$BP_{ML} = \left[BP_{75} + \lambda_{BP} \times \left(75 - FFS_{adj}\right)\right] \times CAF^2$$
$$BP_{ML} = [500 + 0 \times (75 - 60)] \times (1.00)^2 = 500 \text{ pc/h/ln}$$

Because the managed lane flow rate is higher than the breakpoint, three speeds, S_1, S_2, and S_3, need to be computed by using Equations 12-15, 12-17, and 12-19, respectively (with parameters from Exhibit 12-30), as follows:

$$S_1 = FFS_{adj} - A_1 \times \min(v_p, BP) = 60 - 0 \times \min(1{,}519, 500) = 60 \text{ mi/h}$$

$$S_2 = \frac{\left(S_{1,BP} - \dfrac{c_{adj}}{K_c^{nf}}\right)}{\left(c_{adj} - BP\right)^{A_2}}(v_p - BP)^{A_2}$$

$$S_2 = \frac{\left(60 - \dfrac{1{,}650}{30}\right)}{(1{,}650 - 500)^{2.5}}(1{,}519 - 500)^{2.5} = 3.7 \text{ mi/h}$$

$$S_3 = \frac{\left(\dfrac{c_{adj}}{K_c^{nf}}\right) - \left(\dfrac{c_{adj}}{K_c^{f}}\right)}{\left(c_{adj} - BP\right)^2}(v_p - BP)^2$$

$$S_3 = \frac{\left(\dfrac{1{,}650}{30}\right) - \left(\dfrac{1{,}650}{45}\right)}{(1{,}650 - 500)^2}(1{,}519 - 500)^2 = 14.4 \text{ mi/h}$$

The space mean speed of the managed lane is given by Equation 12-12.

$$S_{ML} = \begin{cases} S_1 & v_p \le BP \\ S_1 - S_2 - I_c \times S_3 & BP < v_p \le c \end{cases}$$

Because the managed lane's demand flow of 1,519 pc/h/ln is greater than the breakpoint value of 500 pc/h/ln calculated in Step 4, the second of the two equations applies. To apply this equation, the value of the indicator variable I_c must first be determined from Equation 12-18.

$$I_c = \begin{cases} 0 & K_{GP} \le 35 \text{ pc/mi/ln} \\ & \text{or segment type is Buffer 2, Barrier 1, or Barrier 2} \\ 1 & \text{otherwise} \end{cases}$$

In Case 1, the density of the adjacent general purpose lane is less than 35 pc/mi/ln, as determined in Step 5. As a result, the indicator variable I_c will have a value of zero. Thus, the managed lane speed in Case 1 will be

$$S_{ML,\text{Case1}} = 60 - 3.7 - (0 \times 14.4) = 56.3 \text{ mi/h}$$

In Case 2, the density of the adjacent general purpose lane is greater than 35 pc/ln/mi, and therefore the indicator variable I_c will have a value of 1. The managed lane speed in Case 2 will be

$$S_{ML,\text{Case2}} = 60 - 3.7 - (1 \times 14.4) = 41.9 \text{ mi/h}$$

The managed lane density for the two cases is given by Equation 12-11.

$$D_{ML,\text{Case1}} = \frac{v_p}{S} = \frac{1{,}519}{56.3} = 27.0 \text{ pc/mi/ln}$$

$$D_{ML,\text{Case2}} = \frac{v_p}{S} = \frac{1{,}519}{41.9} = 36.3 \text{ pc/mi/ln}$$

Step 6: Determine LOS

The managed lane facility's density of 27.0 pc/mi/ln under Case 1 corresponds to LOS D, but it is close to the LOS C boundary, which has a maximum value of 26 pc/mi/ln. In Case 2, the density of 36.3 pc/mi/ln corresponds to LOS E.

Discussion

In this example, the managed lane's operating speed and density have been investigated for two operating conditions in the general purpose lanes. When high-density conditions exist in the general purpose lanes, the managed lane's operational speed is reduced and, as a consequence, the managed lane operates at a worse LOS than when lower-density conditions exist in the general purpose lanes.

7. TWO-LANE HIGHWAY EXAMPLE PROBLEMS

Exhibit 26-17 lists the five example problems provided in this section. The problems demonstrate the computational steps involved in applying the two-lane highway automobile and bicycle methodologies.

Problem Number	Description	Type of Analysis
1	Class I highway LOS	Operational analysis
2	Class II highway LOS	Operational analysis
3	Class III highway LOS	Operational analysis
4	LOS for a Class I highway with a passing lane	Operational analysis
5	Two-lane highway bicycle LOS	Planning analysis

The truck analysis methodology for two-lane highways is different from that for basic freeway segments and multilane highways. The methodology for two-lane highways is described in Chapter 15. Among other things, it distinguishes between trucks and recreational vehicles (RVs).

EXAMPLE PROBLEM 1: CLASS I HIGHWAY LOS

The Facts

A segment of Class I two-lane highway has the following known characteristics:

- Demand volume = 1,600 veh/h (total in both directions);
- Directional split (during analysis period) = 50/50;
- PHF = 0.95;
- 50% no-passing zones in the analysis segment (both directions);
- Rolling terrain;
- 14% trucks, 4% RVs;
- 11-ft lane widths;
- 4-ft usable shoulders;
- 20 access points/mi;
- 60-mi/h base FFS; and
- 10-mi segment length.

Find the expected LOS in each direction on the two-lane highway segment as described.

Comments

The problem statement calls for finding the LOS in each direction on a segment in rolling terrain. Because the directional split is 50/50, the solution in one direction will be the same as the solution in the other direction, so only one operational analysis needs to be conducted. The result will apply equally to each direction.

Because this is a Class I highway, both average travel speed (ATS) and percent time spent following (PTSF) must be estimated to determine the expected LOS.

Step 1: Input Data

All input data are specified above.

Step 2: Estimate the FFS

FFS is estimated with Equation 15-2 and adjustment factors found in Exhibit 15-7 (for lane and shoulder width) and Exhibit 15-8 (for access points in both directions). For 11-ft lane widths and 4-ft usable shoulders, the adjustment factor f_{LS} for these features is 1.7 mi/h; for 20 access points/mi, the adjustment factor f_A is 5.0 mi/h. Then

$$FFS = BFFS - f_{LS} - f_A$$
$$FFS = 60.0 - 1.7 - 5.0 = 53.3 \text{ mi/h}$$

Step 3: Demand Adjustment for ATS

The demand volume must be adjusted to a flow rate (in passenger cars per hour) under equivalent base conditions. This adjustment is accomplished with Equation 15-3.

$$v_{i,ATS} = \frac{V_i}{PHF \times f_{g,ATS} \times f_{HV,ATS}}$$

Because the demand split is 50/50, both the analysis direction and opposing demand volumes are 1,600/2 = 800 veh/h.

The grade adjustment factor $f_{g,ATS}$ is selected from Exhibit 15-9 for rolling terrain. The table is entered with a demand flow rate v_{vph} in vehicles per hour, or 800/0.95 = 842 veh/h. By interpolation in Exhibit 15-9 between 800 and 900 veh/h, the factor is 0.99 to the nearest 0.01.

The PCE for trucks and RVs is obtained from Exhibit 15-11 for a demand flow rate of 842 veh/h. Again, by interpolation between 800 and 900 veh/h, the values obtained are $E_T = 1.4$ and $E_R = 1.1$. The heavy-vehicle adjustment is then computed with Equation 15-4.

$$f_{HV,ATS} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$
$$f_{HV,ATS} = \frac{1}{1 + 0.14(1.4 - 1) + 0.04(1.1 - 1)}$$
$$f_{HV,ATS} = 0.943$$

then

$$v_{d,ATS} = v_{o,ATS} = \frac{800}{0.95 \times 0.99 \times 0.943} = 902 \text{ pc/h}$$

Step 4: Estimate ATS

ATS is estimated with Equation 15-6. The adjustment factor $f_{np,ATS}$ is found in Exhibit 15-15 for an FFS of 53.3 mi/h, 50% no-passing zones, and an opposing demand flow of 902 veh/h. This selection must use interpolation on all three scales. Note that interpolation is only to the nearest 0.1 for this adjustment factor. Exhibit 26-18 illustrates the interpolation.

v_o (veh/h)	Factor for FFS = 55 mi/h			Factor for FFS = 50 mi/h		
	40% NPZ	50% NPZ	60% NPZ	40% NPZ	50% NPZ	60% NPZ
800	0.7	*0.9*	1.1	0.6	*0.75*	0.9
902		**0.8**			**0.65**	
1,000	0.6	*0.7*	0.8	0.4	*0.55*	0.7

Notes: $f_{np,ATS}$ = 0.65 + (0.8 - 0.65) (3.3/5.0) = 0.749 = **0.7**.
NPZ = no-passing zones.

Equation 15-6 gives the following:

$$ATS = FFS - 0.00776(v_{d,ATS} + v_{o,ATS}) - f_{np,ATS}$$

$$ATS = 53.3 - 0.00776(902 + 902) - 0.7$$

$$ATS = 53.3 - 14.0 - 0.7 = 38.6 \text{ mi/h}$$

Step 5: Demand Adjustment for PTSF

The adjusted demand used to estimate PTSF is found with Equation 15-7 and Equation 15-8. The grade adjustment factor is taken from Exhibit 15-16 for rolling terrain and a demand flow rate of 800/0.95 = 842 pc/h. PCEs for trucks and RVs are taken from Exhibit 15-18. In both exhibits, the demand flow rate of 842 pc/h is interpolated between 800 pc/h and 900 pc/h to obtain the correct values. The following values are obtained:

$f_{g,PTSF}$ = 1.00

E_T = 1.0

E_R = 1.0

Equation 15-8 gives the following:

$$f_{HV,PTSF} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV,PTSF} = \frac{1}{1 + 0.14(1.0 - 1) + 0.04(1.0 - 1)}$$

$$f_{HV,PTSF} = 1.00$$

and Equation 15-7 gives

$$v_{i,PTSF} = \frac{V_i}{PHF \times f_{g,PTSF} \times f_{HV,PTSF}}$$

$$v_{i,PTSF} = \frac{800}{0.95 \times 1.00 \times 1.00}$$

$$v_{i,PTSF} = 842 \text{ pc/h}$$

Step 6: Estimate PTSF

PTSF is estimated with Equation 15-9 and Equation 15-10. Exhibit 15-20 is used to obtain exponents *a* and *b* for Equation 15-10, and Exhibit 15-21 is used to obtain the no-passing-zone adjustment for Equation 15-9. All three values require interpolation.

Exponents *a* and *b* are based on the opposing flow rate of 842 pc/h, which is interpolated between tabulated values of 800 and 1,000 pc/h. These values are illustrated in Exhibit 26-19.

Opposing Flow Rate (pc/h)	*a*	*b*
800	−0.0045	0.833
842	**−0.0046**	**0.832**
1,000	−0.0049	0.829

Equation 15-10 gives

$$BPTSF = 100[1 - \exp(av_d{}^b)]$$

$$BPTSF = 100[1 - \exp(0.0046 \times 842^{0.832})]$$

$$BPTSF = 71.3\%$$

The adjustment factor for no-passing zones must also be interpolated in two variables. Exhibit 15-21 is entered with 50% no-passing zones, a 50/50 directional split of traffic, and a total two-way demand flow rate of 842 + 842 = 1,684 pc/h. The interpolation is illustrated in Exhibit 26-20.

Total Flow Rate (pc/h)	Adjustment Factor for 40% NPZ	Adjustment Factor for 50% NPZ	Adjustment Factor for 60% NPZ
1,400	23.8	*25.0*	26.2
1,684		16.6 + (25.0 − 16.6) (316/600) = **21.0**	
2,000	15.8	*16.6*	17.4

Note: NPZ = no-passing zone.

Equation 15-9 gives

$$PTSF = BPTSF + f_{np,PTSF}\left(\frac{v_{d,PTSF}}{v_{d,PTSF} + v_{o,PTSF}}\right)$$

$$PTSF = 71.3 + 21.0\left(\frac{842}{842 + 842}\right)$$

$$PTSF = 81.8\%$$

Step 7: Estimate PFFS

This step, which estimates percent of FFS (PFFS), is only used for Class III highways.

Step 8: Determine LOS and Capacity

LOS is determined by comparing the estimated values of ATS and PTSF with the criteria of Exhibit 15-3. An ATS of 38.6 mi/h suggests LOS E will exist, and a PTSF of 81.8% also suggests LOS E will exist. Thus, both criteria lead to the conclusion that the segment will operate at LOS E.

Capacity is determined by either Equation 15-12 or Equation 15-13, whichever produces the lower estimate. Note, however, that all adjustment factors for use in these equations are based on a directional flow rate greater than 900 pc/h. Thus, the grade factor will be 1.00 for both ATS and PTSF. The PCE for trucks is 1.3 for ATS and 1.00 for PTSF; the PCE for RVs is 1.1 for ATS and 1.00 for PTSF.

The adjustment factors for heavy vehicles are as follows:

$$f_{HV,ATS} = \frac{1}{1 + 0.14(1.3 - 1) + 0.04(1.1 - 1)} = 0.96$$

$$f_{HV,PTSF} = \frac{1}{1 + 0.14(1.0 - 1) + 0.04(1.0 - 1)} = 1.00$$

and

$$c_{d,ATS} = 1,700 f_{g,ATS} f_{HV,ATS} = 1,700 \times 1.00 \times 0.960 = 1,632 \text{ veh/h}$$
$$c_{d,PTSF} = 1,700 f_{g,PTSF} f_{HV,PTSF} = 1,700 \times 1.00 \times 1.00 = 1,700 \text{ veh/h}$$

Obviously, the first value holds, and the directional capacity of this facility is 1,632 veh/h. Given the 50/50 directional distribution, the two-way capacity of the segment is 1,632 + 1,632 = 3,264 veh/h. Because this capacity exceeds the limiting capacity of 3,200 pc/h, the directional capacity cannot be achieved with a 50/50 directional distribution. A total two-way capacity of 3,200 pc/h would prevail. In terms of prevailing conditions, the capacity would be 3,200 × 1.00 × 0.960 = 3,072 veh/h. With a 50/50 directional split, this value implies a directional capacity of 3,072/2 = 1,536 veh/h.

Discussion

The two-lane highway segment as described is expected to operate poorly, within LOS E. Although demand is only 842/1,536 = 0.55 of capacity, the operation is poor. Both ATS and PTSF are at unacceptable levels (38.6 mi/h and 81.8%, respectively). This solution again highlights the characteristic of two-lane highways of having poor operations at relatively low volume-to-capacity ratios. This segment should clearly be examined for potential improvements.

Given the 50/50 directional split of traffic, results for the second direction would be identical.

EXAMPLE PROBLEM 2: CLASS II HIGHWAY LOS

The Facts

A segment of Class II highway is part of a scenic and recreational route and has the following known characteristics:

- 1,050 veh/h (both directions);
- 70/30 directional split;
- 5% trucks, 7% RVs;
- PHF = 0.85;
- 10-ft lanes and 2-ft shoulders;
- Base FFS = 55.0 mi/h;
- Rolling terrain;
- 10 access points/mi; and
- 60% no-passing zones.

Comments

Computational Steps 3 and 4, which relate to the estimation of average highway speed, will not be included. LOS for Class II highways depends solely on PTSF. The analysis will be conducted for both the 70% direction of flow and the 30% direction of flow. The necessary computations are accomplished by merely reversing the analysis direction and opposing flows.

Step 1: Input Data

All input data are summarized above.

Step 2: Estimate the FFS

FFS is estimated with Equation 15-2. Adjustment factors for lane and shoulder width (Exhibit 15-7) and access points per mile (Exhibit 15-8) are used.

Exhibit 15-7 is entered with 10-ft lanes and 2-ft shoulders. The resulting adjustment is 3.7 mi/h. Exhibit 15-8 is entered with 10 access points/mi. The resulting adjustment is 2.5 mi/h. FFS is then estimated as follows:

$$FFS = 55.0 - 3.7 - 2.5 = 48.8 \text{ mi/h}$$

Steps 3 and 4: Demand Adjustment for ATS and Estimate ATS

Steps 3 and 4 are not required for Class II highways.

Step 5: Demand Adjustment for PTSF

Equation 15-7 and Equation 15-8 are used to adjust analysis direction and opposing demands to flow rates under equivalent base conditions. With a 70/30 split of traffic, the two demands are as follows:

$$V_{70\%} = V_1 = 1{,}050 \times 0.70 = 735 \text{ veh/h}$$
$$V_{30\%} = V_2 = 1{,}050 \times 0.30 = 315 \text{ veh/h}$$

In this solution, directions will be referred to as 1 and 2. Because both directions are to be analyzed, their position as "analysis direction" and "opposing" will depend on which direction is under study.

Adjustment factors both for grades and for heavy vehicles are needed. Exhibit 15-16 (for grades) and Exhibit 15-18 (for heavy vehicles) are entered with a directional flow rate of 735/0.85 = 865 veh/h (Direction 1) and 315/0.85 = 371 veh/h (Direction 2). Interpolation is required in both cases. The following values are obtained:

$f_{g,PTSF}$ = 1.00 (Direction 1), 0.89 (Direction 2)

E_T = 1.0 (Direction 1), 1.6 (Direction 2)

E_R = 1.0 (Direction 1), 1.0 (Direction 2)

The heavy-vehicle adjustment factor for both directions is computed with Equation 15-8.

$$f_{HV,PTSF,1} = \frac{1}{1 + 0.05(1.0 - 1) + 0.07(1.0 - 1)} = 1.00$$

$$f_{HV,PTSF,2} = \frac{1}{1 + 0.05(1.6 - 1) + 0.07(1.0 - 1)} = 0.97$$

The adjusted demand flow rates are computed with Equation 15-7.

$$v_{1,PTSF} = \frac{735}{0.85 \times 1.00 \times 1.00} = 865 \text{ pc/h}$$

$$v_{2,PTSF} = \frac{315}{0.85 \times 0.89 \times 0.97} = 429 \text{ pc/h}$$

Step 6: Estimate PTSF

PTSF is estimated with Equation 15-9 and Equation 15-10 with values a and b taken from Exhibit 15-20 and $f_{np,PTSF}$ taken from Exhibit 15-21.

Exhibit 15-20 is entered with opposing flow rates of 429 pc/h (for Direction 1) and 865 pc/h (for Direction 2). Both values must be interpolated. The resulting values are as follows:

Direction 1: a = –0.0024, b = 0.915

Direction 2: a = –0.0046, b = 0.832

Exhibit 15-21 is entered with the total demand flow rate of 865 + 429 = 1,294 pc/h, a directional split of 70/30, and 60% no-passing zones. Interpolation is required. The factor is the same for both Directions 1 and 2.

$$f_{np,PTSF} = 23.0\%$$

The base PTSF is computed with Equation 15-10.

$$BPTSF_1 = 100[1 - \exp(-0.0024 \times 865^{0.915})] = 68.9\%$$
$$BPTSF_2 = 100[1 - \exp(-0.0046 \times 429^{0.832})] = 51.0\%$$

PTSF for each direction is computed with Equation 15-9.

$$PTSF_1 = 68.9 + 23.0\left(\frac{865}{865 + 429}\right) = 84.3\%$$
$$PTSF_2 = 51.0 + 23.0\left(\frac{429}{429 + 865}\right) = 58.6\%$$

Step 7: Estimate PFFS

Step 7 is only used for Class III highways.

Step 8: Determine LOS and Capacity

LOS is determined by comparing the PTSF values obtained with the criteria of Exhibit 15-3. Applying these criteria reveals that Direction 1 operates at LOS D and Direction 2 operates at LOS C.

By using the adjustment selected for ≥900 veh/h, capacity is computed with Equation 15-13.

$$c_{1,PTSF} = 1,700 \times 1.00 \times 1.00 = 1,700 \text{ veh/h}$$
$$c_{2,PTSF} = 1,700 \times 1.00 \times 1.00 = 1,700 \text{ veh/h}$$

Discussion

The LOS based solely on PTSF is, at best, somewhat marginal on this two-lane highway segment.

The value of capacity must be carefully considered. If the directional capacities were expanded to two-way capacities on the basis of the given demand split, the capacity in the 30% direction would imply a two-way capacity well in excess of the 3,200-pc/h limitation for both directions. Therefore, even though a capacity of 1,700 veh/h is possible in the 30% direction, it could not occur with a 70/30 demand split. In this case, the two-way capacity would be limited by the capacity in the 70% direction and would be 1,700/0.70 = 2,429 veh/h. The practical capacity for

the 30% direction of flow is actually best estimated as 2,429 – 1,700 or 729 veh/h. Given that the 70/30 directional split holds, when the 30% direction reaches a demand flow rate of 729 veh/h, the opposing direction (the 70% side) would be at its capacity.

EXAMPLE PROBLEM 3: CLASS III HIGHWAY LOS

The Facts

A Class III two-lane highway runs through a rural community in level terrain. It has the following known characteristics:

- Demand volume = 900 veh/h (both directions);
- 10% trucks, no RVs;
- Measured FFS = 40 mi/h;
- 12-ft lanes, 6-ft shoulders;
- PHF = 0.88;
- 80% no-passing zones;
- 60/40 directional split;
- 40 access points/mi; and
- Level terrain.

Comments

Because this is a Class III highway, LOS will be based on PFFS. Thus, Steps 5 and 6, which relate to the estimation of PTSF, will not be used.

Step 1: Input Data

All input data are specified above.

Step 2: Estimate FFS

A measured FFS of 40 mi/h is specified.

Step 3: Demand Adjustment for ATS

The total demand volume of 900 veh/h must be separated into two directional flows. Because both directions will be evaluated, directions are labeled 1 and 2.

$$V_1 = 900 \times 0.60 = 540 \text{ veh/h}$$
$$V_2 = 900 \times 0.40 = 360 \text{ veh/h}$$

The adjusted demand flow rate (in passenger cars per hour) under equivalent base conditions is estimated with Equation 15-3. A grade adjustment factor is selected from Exhibit 15-9, and PCEs for trucks are selected from Exhibit 15-11. Both exhibits are entered with a demand flow rate in vehicles per hour.

$$v_1 = 540/0.88 = 614 \text{ veh/h}$$
$$v_2 = 360/0.88 = 409 \text{ veh/h}$$

The following values are selected from Exhibit 15-9 and Exhibit 15-11. In all cases, interpolation is required.

Value	Direction 1	Direction 2
$f_{g,ATS}$	1.00	1.00
E_T	1.1	1.3

Equation 15-4 gives

$$f_{HV,ATS,1} = \frac{1}{1 + 0.10(1.1 - 1)} = 0.99$$

$$f_{HV,ATS,2} = \frac{1}{1 + 0.10(1.3 - 1)} = 0.97$$

and use of Equation 15-3 gives

$$v_{1,ATS} = \frac{540}{0.88 \times 1.00 \times 0.97} = 620 \text{ pc/h}$$

$$v_{2,ATS} = \frac{360}{0.88 \times 1.00 \times 0.97} = 422 \text{ pc/h}$$

Step 4: Estimate ATS

ATS is estimated with Equation 15-6 with an adjustment factor for no-passing zones taken from Exhibit 15-15. The adjustment factor is based on a 40-mi/h FFS and 80% no-passing zones. Interpolating for an opposing demand flow rate of 422 pc/h (Direction 1) and 620 pc/h (Direction 2) gives the following:

$$f_{np,ATS,1} = 2.4 \text{ mi/h}$$
$$f_{np,ATS,2} = 1.6 \text{ mi/h}$$

Then, use of Equation 15-6 gives

$$ATS_1 = 40.0 - 0.00776(620 + 422) - 2.4 = 29.5 \text{ mi/h}$$
$$ATS_2 = 40.0 - 0.00776(422 + 620) - 1.6 = 30.3 \text{ mi/h}$$

Steps 5 and 6: Demand Adjustment for PTSF and Estimate PTSF

Steps 5 and 6 are not used for Class III highways.

Step 7: Estimate PFFS

The LOS for Class III facilities is based on PFFS achieved, or ATS/FFS. For this segment PFFS is as follows:

$$PFFS_1 = 29.5/40.0 = 73.8\%$$
$$PFFS_2 = 30.3/40.0 = 75.8\%$$

Step 8: Determine LOS and Capacity

From Exhibit 15-3, the LOS for Direction 1 is D, and the LOS for Direction 2 is C. The two values of PFFS are close, but the boundary condition between LOS C and D is 0.75. To be LOS C, PFFS must exceed 0.75, and it is just below the threshold in Direction 1 and just above the threshold in Direction 2.

Capacity is evaluated with adjustment factors for ≥900 pc/h in level terrain. This makes all adjustment factors 1.00 (for ATS). Thus, the capacity in either direction is as follows:

$$c_{1,ATS} = c_{2,ATS} = 1,700 \times 1.00 \times 1.00 = 1,700 \text{ veh/h}$$

The two-way capacity values implied are 1,700/0.60 = 2,833 veh/h (Direction 1) and 1,700/0.40 = 4,250 veh/h (Direction 2). Obviously, the implied two-way capacity is the 2,833 veh/h, which suggests the directional capacity in Direction 2 cannot be achieved with a 60/40 demand split. Rather, the directional capacity in Direction 2 occurs when the capacity in Direction 1 occurs, or 2,833 × 0.40 = 1,133 veh/h.

Discussion

This segment of a Class III two-lane highway operates just at the LOS C–D boundary. Depending on the length of the segment and local expectations, this LOS may or may not be acceptable.

EXAMPLE PROBLEM 4: LOS FOR A CLASS I HIGHWAY WITH A PASSING LANE

The Facts

The 10-mi segment of the two-lane highway analyzed in Example Problem 1 will be improved with 2-mi passing lanes (one in each direction), both installed at 1.00 mi from the segment's beginning. The segment without a passing lane has already been analyzed; the results of that analysis are listed below:

- Demand volume = 800 veh/h in each direction;
- Demand flow rate (ATS) = 902 pc/h in each direction;
- Demand flow rate (PTSF) = 842 pc/h in each direction;
- FFS = 53.3 mi/h;
- ATS = 38.6 mi/h;
- PTSF = 81.8%;
- Rolling terrain; and
- PHF = 0.95.

Comments

Because the directional distribution is 50/50, both directions will involve the same computations, and in both cases the passing lane will start 1.00 mi after the beginning of the 10-mi segment and will end 3.00 mi after the beginning of the segment.

Step 1: Conduct an Analysis Without the Passing Lane

Completed as Example Problem 1.

Step 2: Divide the Segment into Regions

Exhibit 26-21 shows the division of the 6-mi segment into regions. The effective downstream length of the passing lane is selected from Exhibit 15-23

(value is different for ATS and PTSF) for a demand flow rate of 800/0.95 = 842 veh/h.

To Determine	L_u (mi)	L_{pl} (mi)	L_{de} (mi) Exhibit 15-23	L_d (mi) Equation 15-19
ATS	1.00	2.00	1.7	5.3
PTSF	1.00	2.00	4.7	2.3

Step 3: Determine the PTSF

PTSF, as affected by the presence of a passing lane, is estimated with Equation 15-20 and an adjustment factor selected from Exhibit 15-26. The adjustment factor $f_{pl,PTSF}$ is 0.62. Then

$$PTSF_{pl} = \frac{PTSF_d\left[L_u + L_d + f_{pl,PTSF}L_{pl} + \left(\frac{1 + f_{pl.PTSF}}{2}\right)L_{de}\right]}{L_t}$$

$$PTSF_{pl} = \frac{81.8\left[1.0 + 2.3 + (0.62 \times 2.00) + \left(\frac{1 + 0.62}{2}\right)4.7\right]}{10}$$

$$PTSF_{pl} = 68.3\%$$

Step 4: Determine the ATS

ATS as affected by the presence of a passing lane is found with Equation 15-22 and an adjustment factor selected from Exhibit 15-28. The adjustment factor selected is 1.11. Then

$$ATS_{pl} = \frac{ATS_d L_t}{L_u + L_d + \left(\frac{L_{pl}}{f_{pl,ATS}}\right) + \left(\frac{2L_{de}}{1 + f_{pl,ATS}}\right)}$$

$$ATS_{pl} = \frac{38.6 \times 10}{1.00 + 5.3 + \left(\frac{2.00}{1.11}\right) + \left(\frac{2 \times 11.7}{1 + 1.11}\right)}$$

$$ATS_{pl} = 39.7 \text{ mi/h}$$

Step 5: Determine the LOS

Exhibit 15-3 shows that the LOS, as determined by PTSF, has improved to D. The LOS determined by ATS remains E. Thus, although PTSF has improved significantly, the ATS has not improved enough to improve the overall LOS, which remains E.

Discussion

Adding a 2-mi passing lane to a 10-mi segment of Class I highway operating at LOS E was insufficient to improve the overall LOS, although the PTSF did improve from 81.8% to 68.3%. It is likely that a longer (or a second) passing lane would be needed to improve the ATS sufficiently to result in LOS C or LOS D.

EXAMPLE PROBLEM 5: TWO-LANE HIGHWAY BICYCLE LOS

A segment of two-lane highway (without passing lanes) is being evaluated for potential widening, realigning, and repaving. Analyze the impacts of the proposed project on the bicycle LOS (BLOS) in the peak direction.

The Facts

The roadway currently has the following characteristics:

- Lane width = 12 ft,
- Shoulder width = 2 ft,
- Pavement rating = 3 (fair),
- Posted speed limit = 50 mi/h,
- Hourly directional volume = 500 veh/h (no growth is expected),
- Percentage of heavy vehicles = 5%,
- PHF = 0.90, and
- No on-highway parking.

The proposed roadway design has the following characteristics:

- Lane width = 12 ft,
- Shoulder width = 6 ft,
- Pavement rating = 5 (very good),
- Posted speed limit = 55 mi/h, and
- No on-highway parking.

Step 1: Gather Input Data

All data needed to perform the analysis are listed above.

Step 2: Calculate the Directional Flow Rate in the Outside Lane

Using the hourly directional volume and the PHF, calculate the directional demand flow rate with Equation 15-24. Because this is a two-lane highway segment without a passing lane, the number of directional lanes N is 1. Because traffic volumes are not expected to grow over the period of the analysis, v_{OL} is the same for both current and future conditions.

$$v_{OL} = \frac{V}{PHF \times N} = \frac{500}{0.90 \times 1} = 556 \text{ veh/h}$$

Step 3: Calculate the Effective Width

For current conditions, the hourly directional demand V is greater than 160 veh/h and the paved shoulder width is 2 ft; therefore, Equation 15-27 and Equation 15-28 are used to determine the effective width of the outside lane. Under future conditions, the paved shoulder width will increase to 6 ft; therefore, Equation 15-26 and Equation 15-28 are used.

For current conditions,

$$W_v = W_{OL} + W_s = 12 + 2 = 14 \text{ ft}$$

$$W_e = W_v + (\%OHP[2\ ft + W_s]) = 14 + (0 \times [2 + 2]) = 14 \text{ ft}$$

Under the proposed design,

$$W_v = W_{OL} + W_s = 12 + 6 = 18 \text{ ft}$$

$$W_e = W_v + W_s - 2 \times (\%OHP[2 \text{ ft} + W_s]) = 18 + 6 - 2 \times (0 \times [2 + 6]) = 24 \text{ ft}$$

Step 4: Calculate the Effective Speed Factor

Equation 15-30 is used to calculate the effective speed factor. Under current conditions,

$$S_t = 1.1199 \ln(S_p - 20) + 0.8103 = 1.1199 \ln(50 - 20) + 0.8103 = 4.62$$

Under the proposed design,

$$S_t = 1.1199 \ln(55 - 20) + 0.8103 = 4.79$$

Step 5: Determine the LOS

Equation 15-31 is used to calculate the BLOS score, which is then used in Exhibit 15-4 to determine the LOS. Under existing conditions,

$$BLOS = 0.507 \ln(v_{OL}) + 0.1999 S_t (1 + 10.38 HV)^2 + 7.066(1/P)^2 - 0.005(W_e)^2 + 0.760$$

$$BLOS = 0.507 \ln(556) + 0.1999(4.62)(1 + 10.38 \times 0.05)^2 + 7.066(1/3)^2 - 0.005(14)^2 + 0.760$$

$$BLOS = 3.205 + 2.131 + 0.785 - 0.980 + 0.760 = 5.90$$

Therefore, the BLOS for existing conditions is LOS F. Use of the same process for the proposed design results in the following:

$$BLOS = 0.507 \ln(556) + 0.1999(4.79)(1 + 10.38 \times 0.05)^2 + 7.066(1/5)^2 - 0.005(24)^2 + 0.760$$

$$BLOS = 3.205 + 2.209 + 0.283 - 2.880 + 0.760 = 3.58$$

The corresponding LOS for the proposed design is LOS D, close to the boundary of LOS C ($BLOS = 3.50$).

Discussion

Although the posted speed would increase as a result of the proposed design, this negative impact on bicyclists would be more than offset by the proposed shoulder widening, as indicated by the improvement from LOS F to LOS D.

8. REFERENCES

1. Zegeer, J. D., M. A. Vandehey, M. Blogg, K. Nguyen, and M. Ereti. *NCHRP Report 599: Default Values for Highway Capacity and Level of Service Analyses*. Transportation Research Board of the National Academies, Washington, D.C., 2008.

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3. Washburn, S. S., and S. Ozkul. *Heavy Vehicle Effects on Florida Freeways and Multilane Highways*. Report TRC-FDOT-93817-2013. Florida Department of Transportation, Tallahassee, 2013.

4. Ozkul, S., and Washburn, S. S. Updated Commercial Truck Speed versus Distance-Grade Curves for the *Highway Capacity Manual.* In *Transportation Research Record: Journal of the Transportation Research Board, No. 2483*, Transportation Research Board of the National Academies, Washington, D.C., 2015, pp. 91–101.

5. Hu, J., B. Schroeder, and N. Rouphail. Rationale for Incorporating Queue Discharge Flow into *Highway Capacity Manual* Procedure for Analysis of Freeway Facilities. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2286*, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 76–83.

6. Elefteriadou, L., A. Kondyli, and B. St. George. *Estimation of Capacities on Florida Freeways.* Final Report. Transportation Research Center, University of Florida, Gainesville, Sept. 2014.

7. Brilon, W., J. Geistefeldt, and M. Regler. Reliability of Freeway Traffic Flow: A Stochastic Concept of Capacity. In *Proceedings of the 16th International Symposium on Transportation and Traffic Theory*, College Park, Md., July 2005, pp. 125–144.

Some of these references can be found in the Technical Reference Library in Volume 4.

# APPENDIX A: TRUCK PERFORMANCE CURVES

This appendix provides travel time versus distance curves for SUTs and TTs for 50-, 55-, 60-, 65-, and 75-mi/h free-flow speeds (FFS). Curves for SUTs and TTs for a 70-mi/h FFS are presented in Section 3 as Exhibit 26-5 and Exhibit 26-6, respectively.

Exhibit 26-A1
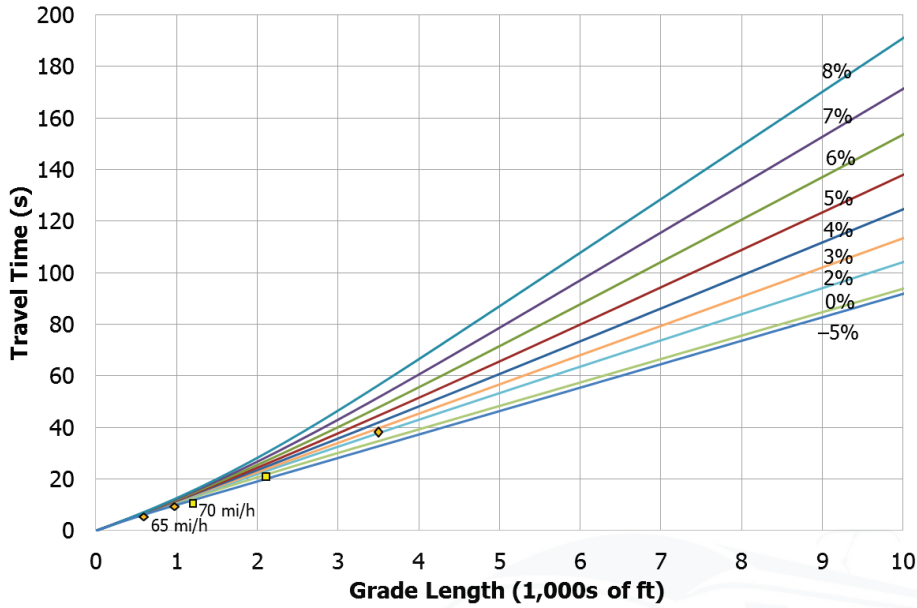SUT Travel Time Versus Distance Curves for 50-mi/h FFS
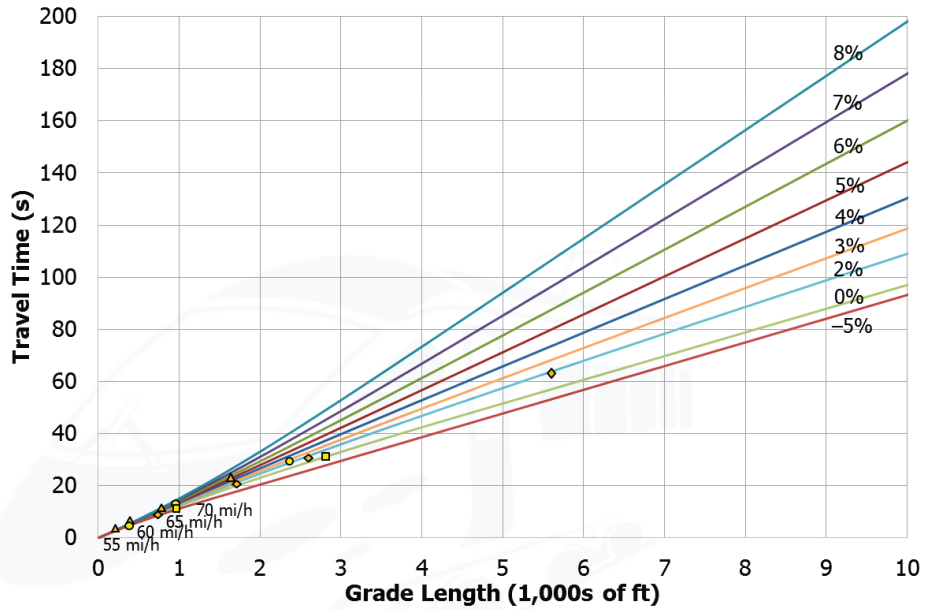
Notes: Curves in this graph assume a weight-to-horsepower ratio of 100.
Triangles indicate where a truck reaches 55 mi/h, circles indicate 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 26-A2
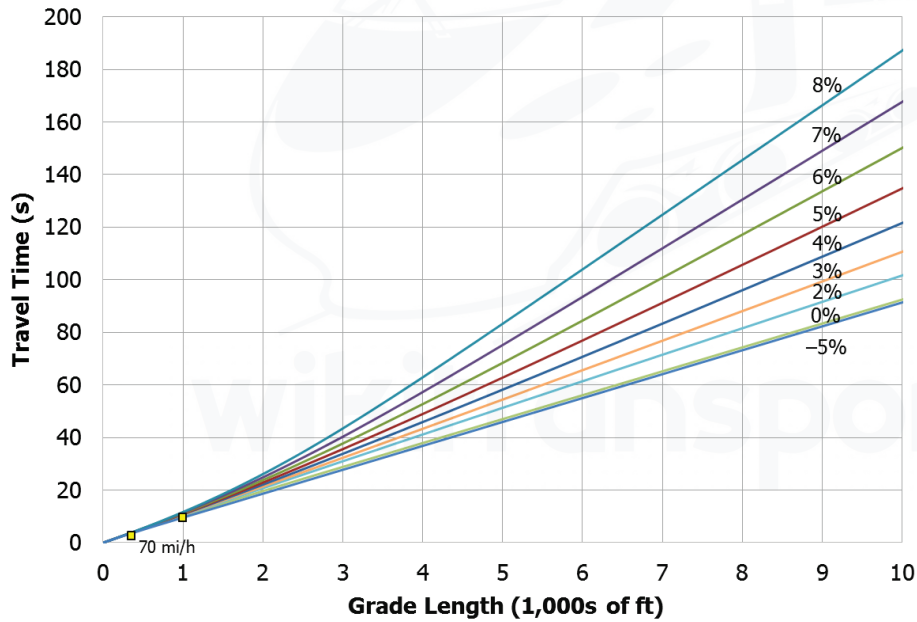SUT Travel Time Versus Distance Curves for 55-mi/h FFS
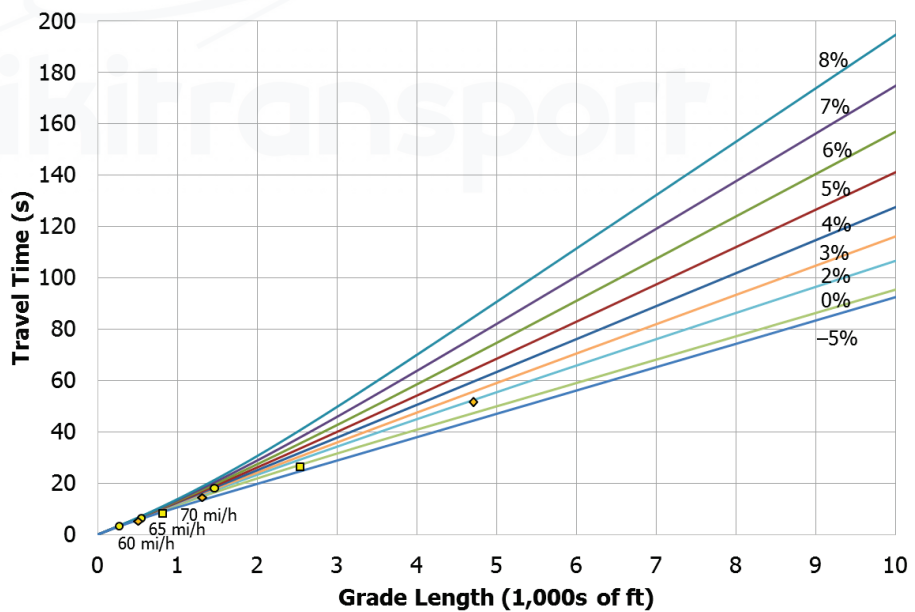
Notes: Curves in this graph assume a weight-to-horsepower ratio of 100.
Circles indicate where a truck reaches 60 mi/h, diamonds indicate 65 mi/h, and squares indicate 70 mi/h.

Exhibit 26-A3
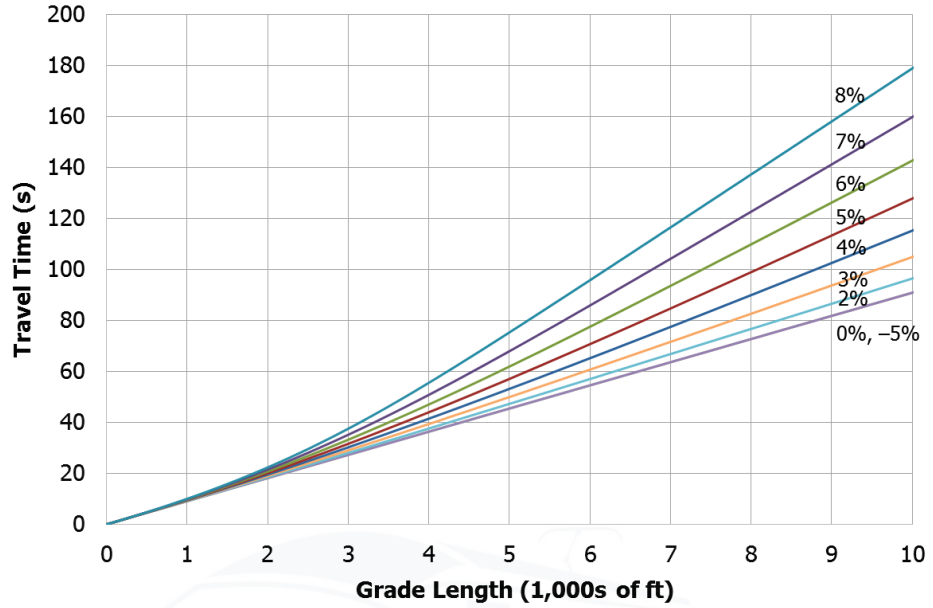SUT Travel Time Versus Distance Curves for 60-mi/h FFS

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 26-A4
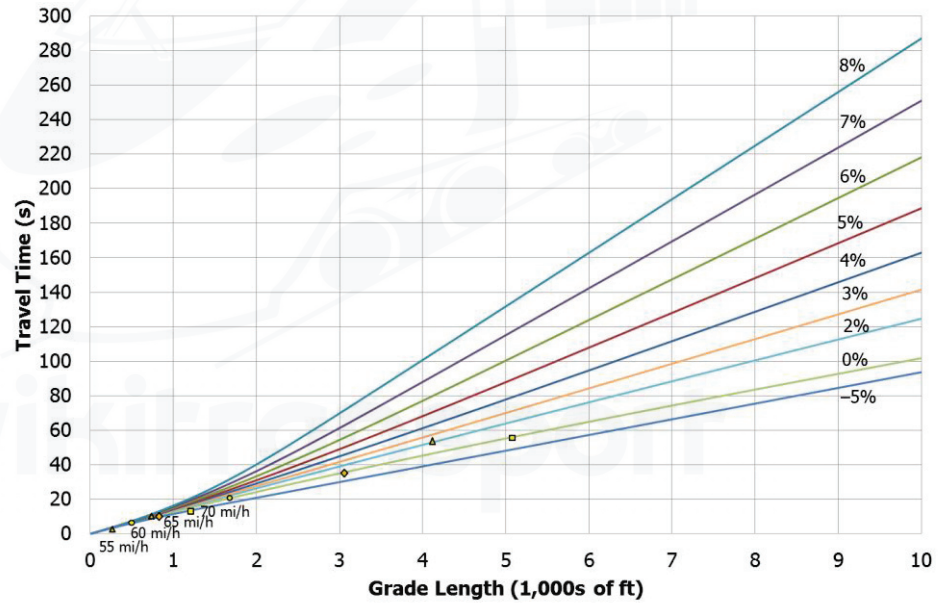SUT Travel Time Versus Distance Curves for 65-mi/h FFS

Notes: Curves in this graph assume a weight-to-horsepower ratio of 100.
Squares indicate where a truck reaches 70 mi/h.

Exhibit 26-A5
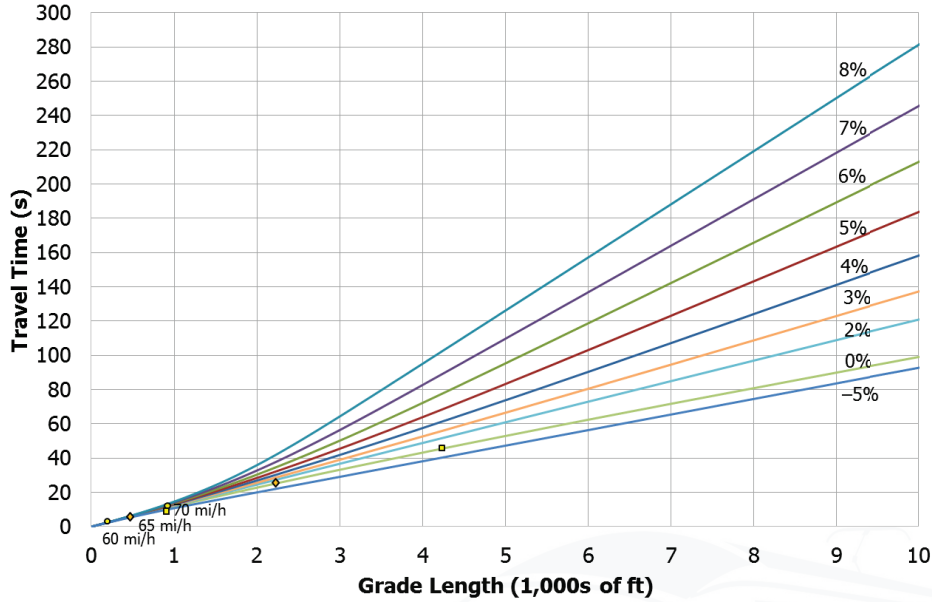SUT Travel Time Versus Distance Curves for 75-mi/h FFS

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

Exhibit 26-A6
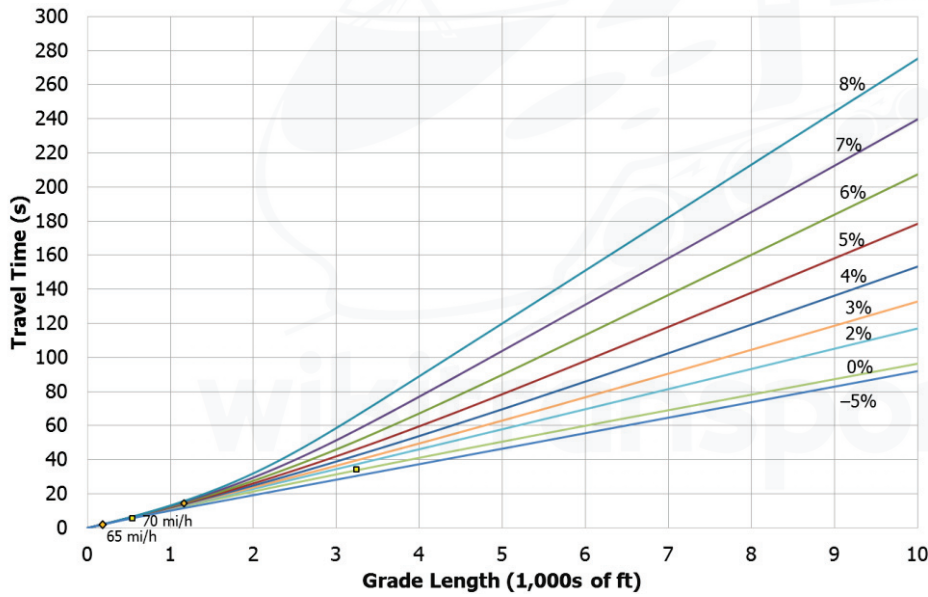TT Travel Time Versus Distance Curves for 50-mi/h FFS

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 26-A7
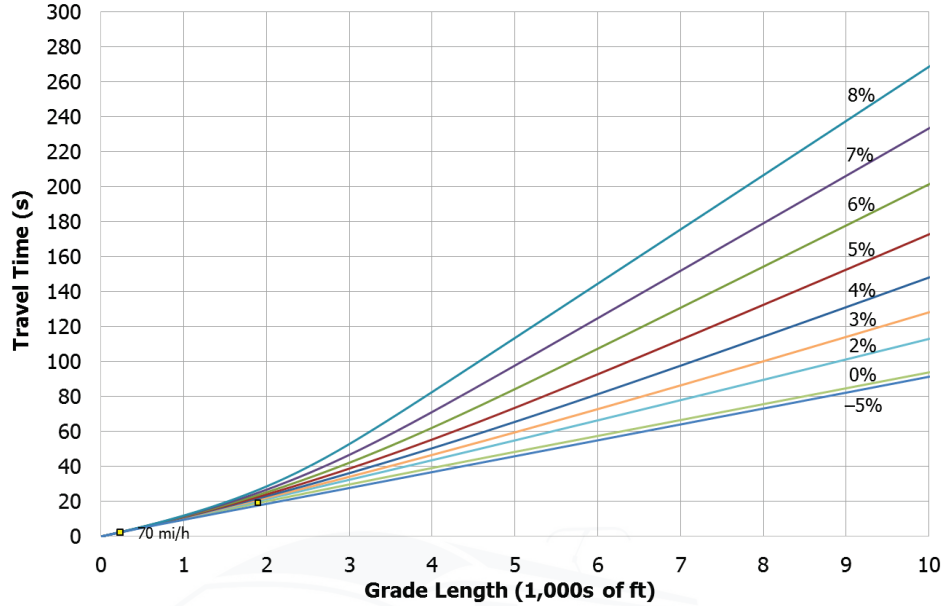TT Travel Time Versus Distance Curves for 55-mi/h FFS

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.

Exhibit 26-A8
TT Travel Time Versus Distance Curves for 60-mi/h FFS

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 26-A9
TT Travel Time Versus Distance Curves for 65-mi/h FFS

Notes: Curves in this graph assume a weight-to-horsepower ratio of 150.
Squares indicate where a truck reaches 70 mi/h.

Exhibit 26-A10
TT Travel Time Versus Distance Curves for 75-mi/h FFS
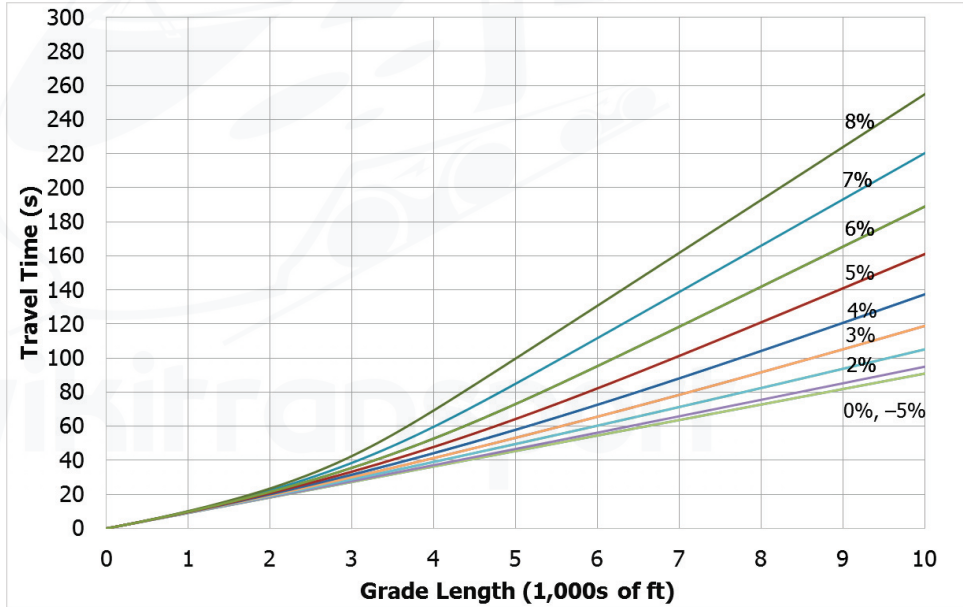
Note: Curves in this graph assume a weight-to-horsepower ratio of 150.

APPENDIX B: WORK ZONES ON TWO-LANE HIGHWAYS

This appendix presents a method for estimating the capacity and operation of work zones on two-lane highways when one of the two lanes is closed. This method is based on research conducted by National Cooperative Highway Research Program (NCHRP) Project 03-107 (*B-1*). At the time of writing, the HCM's two-lane highway methodology was being updated as part of NCHRP Project 17-65 (*B-2*), and it is anticipated that this work zone method will be integrated into the new two-lane highway methodology as part of that work.

Work zones along two-lane highways can take three forms:

1. *Shoulder closure.* Work activity is limited to the shoulder of one direction of travel and does not require lane reconfiguration. In this case, only the direction of travel adjacent to the work zone is slightly affected.

2. *Lane shift.* Work activity extends beyond the shoulder, but both directions of travel can be accommodated with a lane shift that utilizes the opposite paved shoulder.

3. *Lane closure.* Work activity requires the closure of one of the two lanes. Flaggers or temporary traffic signals are used to alternately serve one direction of travel at a time. Both directions of travel can be significantly affected.

The method presented in this appendix addresses the third scenario—lane closure—as it has the greatest impact on traffic operations.

This method addresses a one-lane closure on a two-lane highway. Other types of work zones, such as shoulder closures or lane shifts, are not addressed.

CONCEPTS

A lane closure on a two-lane highway converts traffic flow from an uninterrupted to an interrupted condition. With traffic control devices (flaggers or signals) provided at each end, the operation of the lane closure can be described in terms similar to those used for a signalized intersection:

- *Capacity* is the number of vehicles that can be processed through the work zone per cycle or per hour. It can be determined based on the saturation flow rate at the control points and the traffic control "cycle length."

- *Cycle length* is determined by the flagging operations or signal timing at each control point and the time required to travel through the work zone. Travel time is dependent on the average travel speed (ATS) of the platoons traveling through the work zone. Factors that may influence travel speed include posted speed limit, use of a pilot car, heavy-vehicle percentage, grade, intensity of construction activity, lane width, lateral distance to the work activity, and lighting conditions (day versus night).

Performance measures, including delay and queue length, can be calculated by using capacity and cycle length.

WORK ZONE CAPACITY

The methodology for estimating the capacity of a work zone on a two-lane highway with one lane closed is analogous to the capacity calculation for a two-phase signalized intersection. ATS is estimated from a regression model developed through observations of two directions of travel at three work zones (*B-1*).

Step 1: Collect Data

For a typical capacity calculation, the analyst must specify traffic information (including traffic demands, travel speed, and heavy-vehicle percentage), roadway geometric configuration (e.g., lane width, lateral clearance, speed limit), and work zone data (including work zone length, signal green time, and traffic control plan).

A basic traffic flagger control process for a two-lane highway work zone involving a lane closure is shown in Exhibit 26-B1. Direction 1 refers to the travel direction whose lane is blocked by the work zone; Direction 2 refers to the travel direction with the open lane.
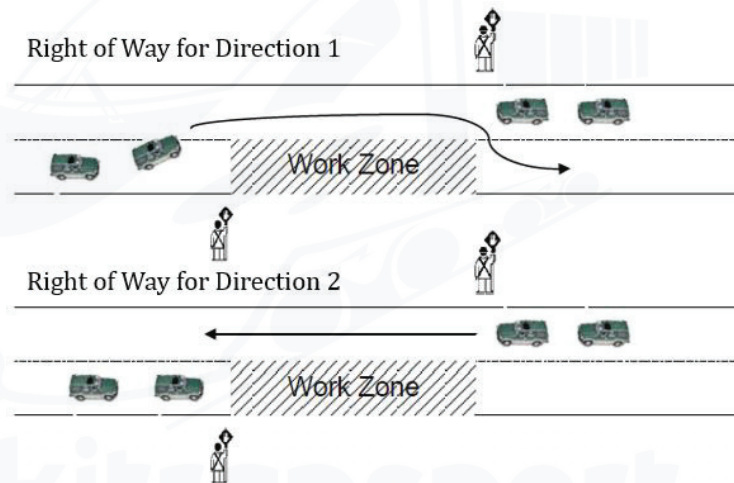
Source: Schoen et al. (*B-1*).

Some data, such as ATS, saturation flow rate, and green interval length, may be difficult to collect in the field. In Steps 2–4, the mathematical models that can be used to estimate these data are presented. Analysts must note that, for capacity calculations, field data are always more desirable to use when available.

A procedure is given in Section 6 of Chapter 31, Signalized Intersections: Supplemental, for determining the saturation flow rate of a signalized intersection. This procedure involves counting and timing the number of queue discharge vehicles that pass through an intersection to determine the saturated vehicle headway. As two-lane highway work zone traffic control typically has a much longer cycle length than a typical signalized intersection, the time period for gathering saturation flow data is recommended to be 30–60 min. Of course, a longer time period is generally more desirable when possible. The work zone capacity can then be determined from the measured saturation flow rate and the effective green–to–cycle length ratio.

Step 2: Estimate ATS

A simple estimation of ATS can be obtained by following a procedure similar to the general procedure described in Chapter 15 for estimating two-lane highway ATS. Speeds for Directions 1 and 2 are calculated by Equation 26-B1 and Equation 26-B2, respectively. Research on two-lane highway work zones (*B-1*) found that Direction 2 (i.e., the direction whose lane is not closed) consistently had higher average speeds than Direction 1.

$$S_1 = 0.615 \times SL - f_{LS} - f_A - f_{np,ATS}$$

$$S_2 = 0.692 \times SL - f_{LS} - f_A - f_{np,ATS}$$

Equation 26-B1

Equation 26-B2

where

S_i = ATS in direction i (mi/h),

SL = speed limit for the two-lane highway segment (mi/h),

f_{LS} = adjustment for lane and shoulder width from Exhibit 15-7 (mi/h),

f_A = adjustment for access-point density from Exhibit 15-8 (mi/h), and

$f_{np,ATS}$ = adjustment factor for ATS determination for the percentage of no-passing zones in the analysis direction (mi/h) = 2.4 mi/h.

For two-lane highway work zones, $f_{np,ATS}$ provides a constant speed reduction of 2.4 mi/h in all conditions.

Step 3: Estimate Saturation Flow Rate

If the saturation flow rate is not measured in the field, a directional saturation flow rate can be estimated by using Equation 26-B3 with Equation 26-B4 and Equation 26-B5.

$$s_i = \frac{3,600}{\hat{h}_i}$$

Equation 26-B3

with

$$\hat{h}_i = h_0 \times f_{\text{speed},i}$$

$$f_{\text{speed},i} = 1 - 0.005(\min[S_i, 45] - 45)$$

Equation 26-B4

Equation 26-B5

where

s_i = saturation flow rate for direction i (pc/h);

$\hat{h}_i$ = adjusted time headway for direction i (s);

h_0 = base saturation headway (s/pc) = 3,600/1,900 = 1.89 s/pc;

$f_{\text{speed},i}$ = ATS adjustment for direction i (decimal); and

S_i = ATS in direction i (mi/h).

Step 4: Estimate Green Time

The length of the green interval can be applied directly if a fixed-time signal is applied at the work zone site. However, most work zones apply flagger control, for which the green time in each cycle is not fixed. For flagger control under relatively balanced directional demand conditions, a simple estimation of optimal directional effective green time can be found by using Equation 26-B6.

Equation 26-B6

$$G_{opt} = \begin{cases} 20 & 0.0375l < 20 \\ 0.0375l & 20 \leq 0.0375l \leq 60 \\ 60 & 0.0375l > 60 \end{cases}$$

where

G_{opt} = optimal effective green time for one direction (s), and

l = work zone length (ft).

To ensure traffic can be fully discharged in two directions, directional effective green-time lengths must satisfy Equation 26-B6

Equation 26-B7

$$G_i \geq G_{i,min} = \frac{v_i}{s_i - v_i}(C - G_i)$$

with

Equation 26-B8

$$C = \frac{l}{S_{1,fps}} + \frac{l}{S_{2,fps}} + G_1 + G_2 + 2L_S$$

where

G_i = effective green time for direction i (s),

$G_{i,min}$ = minimum effective green time for direction i (s),

s_i = saturation flow rate for direction i (pc/h),

v_i = demand flow rate for direction i (pc/h),

C = cycle length (s),

$S_{i,fps}$ = ATS in direction i (ft/s) = (S_i × 5,280 ft/mi)/(3,600 s/h),

S_i = ATS in direction i (mi/h), and

L_S = start-up lost time (s).

Step 5: Calculate Capacity

Directional capacity is calculated by Equation 26-B9.

Equation 26-B9

$$c_i = \frac{s_i G_i}{C}$$

where

c_i = capacity for direction i (pc/h),

s_i = saturation flow rate for direction i (pc/h),

G_i = effective green time for direction i (s), and

C = cycle length (s).

The *start-up lost time*, the elapsed time between the last vehicle in the opposing direction exiting the work zone and the entry of the first queued vehicle traveling in the subject direction, is assumed to be independent of traffic direction, as the two directions follow the same traffic control plan. A default value of 2 s for each direction is recommended.

The total capacity c_{total} (in passenger cars per hour) can be calculated by summing the two directional capacities, as shown in Equation 26-B6.

$$c_{\text{total}} = c_1 + c_2 = \frac{s_1 G_1 + s_2 G_2}{C}$$

Equation 26-B10

QUEUING AND DELAY ANALYSIS

The previous steps provide a simple procedure to check two-lane highway work zone capacity. In practice, it might also be useful to have performance data such as delay and queuing. Users can apply the model to determine the optimal control plan while minimizing the vehicle delay and queuing data.

A simple way to estimate vehicle delay and queue length is by assuming deterministic traffic flow for both directions. Exhibit 26-B2 shows the deterministic queuing diagram for a two-lane highway work zone. Although more accurate estimates can be calculated from microscopic simulations that incorporate random processes, these estimates might be difficult to accomplish in practice because of the extra time and resources required. Therefore, by a similar procedure to that used in Chapter 19 for signalized intersection control delay estimation, the incremental delay caused by random arrivals is added to the deterministic queuing delay associated with the work zone. The interval g_i shown in the exhibit is the portion of the green time with saturated departures.

The maximum queue length for each direction $Q_{i,max}$ (in passenger cars) is the height of the triangles in the queue length area of the exhibit. These lengths can be calculated by Equation 26-B11 and Equation 26-B12 for Directions 1 and 2, respectively.
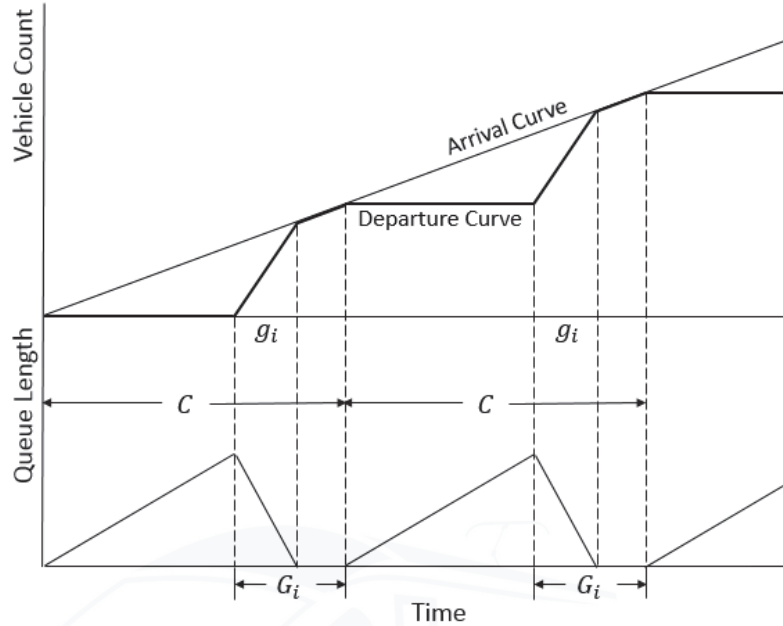
$$Q_{1,max} = \frac{v_1}{3,600}\left(\frac{l}{S_{1,fps}} + \frac{l}{S_{2,fps}} + G_2 + 2L_S\right)$$

Equation 26-B11

$$Q_{2,max} = \frac{v_2}{3,600}\left(\frac{l}{S_{1,fps}} + \frac{l}{S_{2,fps}} + G_1 + 2L_S\right)$$

Equation 26-B12

Exhibit 26-B2
Directional Queueing Diagram for a Two-Lane Highway Lane-Closure Work Zone

Source: Schoen et al. (***Error! Reference source not found.***).

For undersaturated conditions, directional vehicle delay caused by a two-lane highway work zone with one lane closed can be represented by Equation 26-B6

Equation 26-B13

$$d = d_1 + d_2$$

where

d = control delay per passenger car (s/pc),

d_1 = uniform control delay assuming uniform traffic arrivals (s/pc), and

d_2 = incremental delay resulting from random arrivals and oversaturation queues (s/pc).

For each direction i, the total directional uniform control delay per cycle $D_{i,1}$ (in seconds) is the triangle area in the queue length diagram (Exhibit 26-B2). It is calculated as one-half the queue length multiplied by the queueing duration. $D_{i,1}$ is given by Equation 26-B6.

Equation 26-B14

$$D_{1,i} = \frac{s_i v_i}{2(s_i - v_i)}(C - G_i)^2$$

The average uniform delay in direction i is given by Equation 26-B15.

Equation 26-B15

$$d_{1,i} = \frac{D_{1,i}}{v_i C} = \frac{s_i(C - G_i)^2}{2(s_i - v_i)C}$$

Finally, by following Equation 19-26 in Chapter 19, the average incremental delay in direction i is given by Equation 26-B16.

Equation 26-B16

$$d_{2,i} = 900\,T\left[(X_i - 1) + \sqrt{(X_i - 1)^2 + \frac{8kIX_i}{c_i T}}\right]$$

where

T = analysis period duration (h),

k = incremental delay factor (decimal),

I = upstream filtering adjustment factor (decimal),

c_i = directional capacity (pc/h) from Equation 26-B9, and

X_i = directional volume-to-capacity ratio or degree of saturation (unitless).

Values for k can be calculated with Equation 19-22 in Chapter 19. For fixed-time control, $k = 0.5$. Because the purpose of calculating delay in a work zone context is to identify the optimal effective green time, which is assumed to repeat every cycle, a value for k of 0.5 is recommended for use in Equation 26-B16. It incorporates the effects of metered arrivals from upstream signals or work zones. If the work zone is isolated, then $I = 1.0$.

The average delay per passenger car is the sum of the directional total delays, divided by the total number of passenger cars, as shown in Equation 26-B17. Note that the traffic flow rates used in the equation are in units of passenger cars per hour; therefore, vehicle delay is calculated in terms of seconds per passenger car.

$$d = \frac{(d_{1,1} + d_{2,1})v_1 + (d_{1,2} + d_{2,2})v_2}{v_1 + v_2}$$

Equation 26-B17

In equations calculating queue length and vehicle delay, all variables are given by roadway or traffic parameters, except that directional effective green time G_i should be determined by users. Thus users can change the traffic control plan to optimize the result. Users must note, however, that they should not arbitrarily choose an effective green-time value.

EXAMPLE CALCULATION

This subsection presents an example application of the methodology. An isolated 1,000-ft-long work zone will be located on a rural two-lane highway. Known peak hour roadway and traffic parameters are summarized in Exhibit 26-B3 and Exhibit 26-B4.

Direction	Lane Width (ft)	Shoulder Width (ft)	No. of Access Points per Mile	General Terrain Type
1	12	3	0	Rolling
2	12	3	0	Rolling

Exhibit 26-B3
Example Calculation: Work Zone Roadway Parameters

Direction	Speed Limit (mi/h)	Traffic Demand (veh/h)	Truck Percentage	RV Percentage
1	45	300	10.0	10.0
2	45	300	10.0	10.0

Exhibit 26-B4
Example Calculation: Work Zone Traffic Parameters

Step 1: Collect Data

Most of the necessary data are provided in the problem statement. However, for the purposes of calculating ATS, the traffic demand V_i (in vehicles per hour) must be converted into a traffic flow rate $v_{i,ATS}$ (in passenger cars per hour) by using Equation 15-3 in Chapter 15.

$$v_{i,ATS} = \frac{V_i}{PHF \times f_{g,ATS} \times f_{HV,ATS}}$$

This equation requires determining both an adjustment factor for grade (in this case, general terrain) and an adjustment factor for heavy vehicles (which also includes terrain effects). In addition, a peak hour factor (PHF) is applied.

As the PHF for this highway is not known, the default value of 0.88 given in Exhibit 15-5 will be used. From Exhibit 15-9, the ATS grade adjustment factor $f_{g,ATS}$ for rolling terrain is 0.83. Finally, from Exhibit 15-11, the truck PCE for ATS calculation purposes in rolling terrain is 2.1, and the RV PCE is 1.1. The ATS heavy vehicle adjustment factor $f_{HV,ATS}$ can then be calculated from Equation 15-4.

$$f_{HV,ATS} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$
$$f_{HV,ATS} = \frac{1}{1 + (0.10)(2.1 - 1) + (0.10)(1.1 - 1)}$$
$$f_{HV,ATS} = 0.89$$

then

$$v_1 = v_2 = \frac{300}{0.88 \times 0.83 \times 0.89} = 461 \text{ pc/h}$$

Step 2: Estimate ATS

ATS through the work zone is calculated with Equation 26-B1 and Equation 26-B2 for Directions 1 and 2, respectively.

$$S_1 = 0.615 \times SL - f_{LS} - f_A - f_{np,ATS}$$
$$S_2 = 0.692 \times SL - f_{LS} - f_A - f_{np,ATS}$$

The speed limit SL is given, and the ATS adjustment factor for the percentage of no-passing zones in the analysis direction is a constant of 2.4 mi/h according to the text accompanying Equation 26-B2. From Exhibit 15-7, the adjustment for lane and shoulder width f_{LS} is 2.6 mi/h for 12-ft lane widths and 3-ft shoulder widths. Finally, from Exhibit 15-8, the adjustment for access point density is 0.0 mi/h when no access points are present. Then

$$S_1 = 0.615 \times 45 - 2.6 - 0 - 2.4 = 22.7 \text{ mi/h}$$
$$S_2 = 0.692 \times 45 - 2.6 - 0 - 2.4 = 26.1 \text{ mi/h}$$

Step 3: Estimate Saturation Flow Rate

Equation 26-B3 through Equation 26-B5 are used to estimate the saturation flow rate through the work zone.

First, the speed adjustment factor is calculated for each direction as follows:

$$f_{\text{speed},i} = 1 - 0.005(\min[S_i, 45] - 45)$$

$$f_{\text{speed},1} = 1 - 0.005(\min[22.7, 45] - 45) = 1.11$$

$$f_{\text{speed},2} = 1 - 0.005(\min[26.1, 45] - 45) = 1.09$$

Next, an adjusted time headway is calculated for each direction as follows:

$$\widehat{h_i} = h_0 \times f_{\text{speed},i}$$

$$\widehat{h_1} = 1.89 \times 1.11 = 2.10 \text{ s}$$

$$\widehat{h_2} = 1.89 \times 1.09 = 2.06 \text{ s}$$

where the base saturation headway of 1.89 s/pc is as given in the text following Equation 26-B4.

Finally, the saturation flow rate for each direction is calculated as

$$s_i = \frac{3,600}{\widehat{h_i}}$$

$$s_1 = \frac{3,600}{2.10} = 1,714 \text{ pc/h/ln}$$

$$s_2 = \frac{3,600}{2.06} = 1,748 \text{ pc/h/ln}$$

Step 4: Estimate Green Time

In Step 4, the effective green time length is determined. It may be difficult to choose a green time value without knowing the traffic performance parameters, but an estimate of the optimal value can be obtained with Equation 26-B6.

$$G_{opt} = \begin{cases} 20 & 0.0375l < 20 \\ 0.0375l & 20 \leq 0.0375l \leq 60 \\ 60 & 0.0375l > 60 \end{cases}$$

As the work zone will be 1,000 ft long, the value $0.0375l$ computes to 37.5 s. As 37.5 is between 20 and 60, it can be used directly; however, this value should be checked to make sure it is long enough to discharge the vehicle queues. Equation 26-B7 provides this check.

$$G_i \geq G_{i,min} = \frac{v_i}{s_i - v_i}(C - G_i)$$

The cycle length C is computed from Equation 26-B8, incorporating a default value of 2.0 s for the start-up lost time.

$$C = \frac{l}{S_{1,fps}} + \frac{l}{S_{2,fps}} + G_1 + G_2 + 2L_S$$

$$C = \frac{1,000}{22.7 \times 5,280/3,600} + \frac{1,000}{26.1 \times 5,280/3,600} + 37.5 + 37.5 + 2(2.0)$$

$$C = 135.2 \text{ s}$$

then

$$G_{1,min} = \frac{461}{1,714 - 461}(135.2 - 37.5) = 35.9 \text{ s}$$

$$G_{2,min} = \frac{461}{1,748 - 461}(135.2 - 37.5) = 35.0 \text{ s}$$

As the optimal effective green time of 37.5 s is greater than the minimum required time for each direction, it is accepted, and the process continues to Step 5.

Step 5: Calculate Capacity

Directional capacity is calculated with Equation 26-B9.

$$c_i = \frac{s_i G_i}{C}$$

$$c_1 = \frac{(1{,}714)(37.5)}{135.2} = 475 \text{ pc/h}$$

$$c_2 = \frac{(1{,}748)(37.5)}{135.2} = 485 \text{ pc/h}$$

As $v_1 < c_1$ and $v_2 < c_2$, this 1,000-ft work zone can serve the traffic demand without accumulating vehicle queues when the effective green time is 37.5 s for both directions.

Queuing and Delay

If desired, the maximum queue length and average vehicle delay can be calculated for both directions. The maximum queue length is calculated from Equation 26-B11 and Equation 26-B12 for Directions 1 and 2, respectively.

$$Q_{1,max} = \frac{v_1}{3{,}600}\left(\frac{l}{S_{1,fps}} + \frac{l}{S_{2,fps}} + G_2 + 2L_S\right)$$

$$Q_{1,max} = \frac{461}{3{,}600}(30.0 + 26.1 + 37.5 + 4.0) = 13 \text{ veh}$$

$$Q_{2,max} = \frac{v_2}{3{,}600}\left(\frac{l}{S_{1,fps}} + \frac{l}{S_{2,fps}} + G_1 + 2L_S\right)$$

$$Q_{2,max} = \frac{461}{3{,}600}(30.0 + 26.1 + 37.5 + 4.0) = 13 \text{ veh}$$

The average uniform delay by direction is calculated with Equation 26-B15.

$$d_{1,i} = \frac{s_i(C - G_i)^2}{2(s_i - v_i)C}$$

$$d_{1,1} = \frac{(1{,}714)(135.2 - 37.5)^2}{(2)(1{,}714 - 461)(135.2)} = 48.3 \text{ s/veh}$$

$$d_{1,2} = \frac{(1{,}748)(135.2 - 37.5)^2}{(2)(1{,}748 - 461)(135.2)} = 47.9 \text{ s/veh}$$

The average incremental delay by direction is calculated from Equation 26-B16 The recommended value of 0.5 is used for the incremental delay factor k, and as the work zone is isolated, a value of 1.0 is used for the upstream filtering adjustment factor I.

$$d_{2,i} = 900\,T\left[(X_i - 1) + \sqrt{(X_i - 1)^2 + \frac{8kIX_i}{c_i T}}\right]$$

$$d_{2,1} = (900)(1)\left[\left(\frac{461}{475} - 1\right) + \sqrt{\left(\frac{461}{475} - 1\right)^2 + \frac{(8)(0.5)(1.0)\left(\frac{461}{475}\right)}{(475)(1)}}\right] = 59.1 \text{ s}$$

$$d_{2,2} = (900)(1)\left[\left(\frac{461}{485} - 1\right) + \sqrt{\left(\frac{461}{485} - 1\right)^2 + \frac{(8)(0.5)(1.0)\left(\frac{461}{485}\right)}{(485)(1)}}\right] = 46.8 \text{ s}$$

Finally, the average delay per passenger car is given by Equation 26-B17.

$$d = \frac{(48.3 + 59.1)(461) + (47.9 + 46.8)(461)}{461 + 461} = 101 \text{ s}$$

REFERENCES

B-1.   Schoen, J. M., J. A. Bonneson, C. Safi, B. Schroeder, A. Hajbabaie, C. H. Yeom, N. Rouphail, Y. Wang, W. Zhu, and Y. Zou. *Work Zone Capacity Methods for the Highway Capacity Manual.* National Cooperative Highway Research Program Project 3-107 final report, preliminary draft. Kittelson & Associates, Inc., Tucson, Ariz., April 2015.

B-2.   National Cooperative Highway Research Program Project 17-65 Website. http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=3658. Accessed May 28, 2015.