Sketch Methods for Estimating Incident-Related Impacts

user's guide

prepared for

Federal Highway Administration

prepared by

Cambridge Systematics, Inc.

with

Harry Cohen
and
Science Applications International Corporation

December 1998

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prepared for

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Office of Environment and Planning
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prepared by

Cambridge Systematics, Inc.
150 CambridgePark Drive, Suite 4000
Cambridge, MA 02140

with

Harry Cohen
and
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1.0 Introduction

This document serves as a stand-alone guide for planners to use in applying the methodology for studying incident impacts and the effects of incident management strategies. It reformats material found in the Final Report[1] in a more easily followed procedure for actual applications, and also provides an example of how the methodology is applied to a freeway corridor.

Given the complexity of the equations for predicting delay, it is recommended that planners imbed them in a spreadsheet or other suitable software and that they be checked thoroughly before they are applied. Input data can be developed off-line. An example of a spreadsheet format is presented as the tables in Chapter 4.0 for guidance.

[1] *Sketch Methods for Estimating Incident-Related Impacts* (Task Order 21, Contract DTFH-61-95-C-00060). The research for this Task Order was undertaken to develop and validate a sketch planning method for estimating the impacts of non-recurring congestion (incidents) and the effects of strategies to mitigate that congestion.

2.0 Modeling Results

2.1 Prediction Equations for Freeways and Signalized Arterials

The QSIM output from the study was used to fit a series of equations for both daily and peak-period values for H_u, H_i, and H_r. *The analyst is referred to Section 3.0 for a complete discussion of how to develop the independent variables and how to apply the procedure.* The analyst should be particularly aware of the different definitions used for queuing (H_r and H_i) and non-queuing (H_u) factors. The queuing factors account for actual delay due to queues. The non-queuing factor is basically the inverse of speed for the segment and therefore is not pure delay; delay can be obtained by comparing the computed speed to free flow or "desired" speed. (See discussion in Section 3.0). The peak period is defined as weekdays from 6:00 a.m. to 10:00 a.m. and 3:00 p.m. to 7:00 p.m. for both directions of travel. The daily equations include both weekdays and weekends.

A preliminary attempt at developing incident relationships for signalized arterials was also undertaken but the basic incident distributions were borrowed from freeways (with the exception of the accident rate portion). Little is currently known about the characteristics of incidents on signalized arterials and the viability of using the freeway distributions is unknown. Therefore, the arterial equations should be applied judiciously. If anything, the equations probably overestimate delay because, in reality, diversion around arterial incidents is much more possible than on freeways. Limiting traffic flow impacts to just lane blockages helps to reduce the impact, but this was a purely judgmental decision on the part of the Research Team. The equations are as follows.

Table 2.1 Results for Freeways and Signalized Arterials
A.M. Peak Direction, Daily Traffic

Freeways

<u>Travel Time without Queuing (hours per vehicle mile)</u>

$$H_u = 1 / \text{Speed} = (1 / Sf)\,(1 + 5.44E\text{-}12 * X^{10})$$

for X <= 8

$$H_u = 1 / \text{Speed} = (1 / Sf)\,(1.23E{+}00 \quad 7.12E\text{-}02 * X + 6.78E\text{-}03 * X^2 + \quad 1.83E\text{-}04 * X^3)$$

for X > 8

<u>Delay Due to Recurring Queues (hours per bottleneck)</u>

$$H_r = \text{RECURRING DELAY} = 0$$

for X <=8

H_r = RECURRING DELAY = 6.77E-03 * (X 8) 4.13E-03 * (X 8) ^{2} + 1.29E-03 * (X 8) ^{3}

<u>Delay Due to Incidents for Two Lane Facilities (hours per vehicle mile)</u>

H_i = INCIDENT DELAY = IncRate * DurFac^2 * (1 + 4.22 * (1-SF)$^{1.05}$)
* 5.26E-08 * X$^{6.26E+00}$ * e$^{-4.53E-01 * X}$

for X <= 8

H_i = INCIDENT DELAY = IncRate * DurFac^2 * (1 + 4.22 * (1-SF)$^{1.05}$)
* 8.11E-10 * X$^{7.23E+00}$ * e$^{-1.83E-01 * X}$

for X > 8

<u>Delay Due to Incidents for Three Lane Facilities (hours per vehicle mile)</u>

H_i = INCIDENT DELAY = IncRate * DurFac^2 * (1 + 3.77 * (1-SF)$^{1.04}$)
* 1.64E-06 * X$^{-4.34E-01}$ * e$^{8.10E-01 * X}$

for X <= 8

H_i = INCIDENT DELAY = IncRate * DurFac^2 * (1 + 3.77 * (1-SF)$^{1.04}$)
* 3.39E-11 * X$^{9.04E+00}$ * e$^{-3.04E-01 * X}$

for X > 8

<u>Delay Due to Incidents for Four Lane Facilities (hours per vehicle mile)</u>

H_i = INCIDENT DELAY = IncRate * DurFac^2 * (1 + 3.45 * (1-SF)$^{1.04}$)
* 6.90E-08 * X$^{1.27E+00}$ * e$^{7.49E-01 * X}$

for X <= 8

H_i = INCIDENT DELAY = IncRate * DurFac^2 * (1 + 3.45 * (1-SF)$^{1.04}$)
* 1.09E-11 * X$^{9.68E+00}$ * e$^{-3.42E-01 * X}$

for X > 8

<u>Delay Due to Accidents for Two Lane Facilities (hours per vehicle mile)</u>

H_a = ACCIDENT DELAY = (AccRate - 1) * DurFac^2 * (1 + 4.22 * (1-SF)$^{1.05}$)
* 1.15E-06 * X$^{2.63E+00}$ * e$^{5.75E-02 * X}$

for X <= 8

H_a = ACCIDENT DELAY = (AccRate - 1) * DurFac^2 * (1 + 4.22 * (1-SF)$^{1.05}$)
* 1.50E-09 * X$^{6.52E+00}$ * e$^{-1.22E-01 * X}$

for X > 8

<u>Delay Due to Accidents for Three Lane Facilities (hours per vehicle mile)</u>

H_a = ACCIDENT DELAY = (AccRate - 1) * DurFac^2 * (1 + 3.77 * (1-SF)$^{1.04}$)
* 1.36E-07 * X$^{3.09E+00}$ * e$^{1.60E-01 * X}$

for X <= 8

H_a = ACCIDENT DELAY = (AccRate - 1) * DurFac^2 * (1 + 3.77 * (1-SF)$^{1.04}$)
* 6.09E-10 * X$^{6.53E+00}$ * e$^{-5.75E-02 * X}$

for X > 8

<u>Delay Due to Accidents for Four Lane Facilities (hours per vehicle mile)</u>

H_a = ACCIDENT DELAY = (AccRate - 1) * DurFac^2 * (1 + 3.45 * (1-SF)$^{1.04}$)
* 4.31E-08 * X$^{2.82E+00}$ * e$^{3.57E-01 * X}$

for X <= 8

H_a = ACCIDENT DELAY = (AccRate - 1) * DurFac^2 * (1 + 3.45 * (1-SF)$^{1.04}$)
* 1.47E-10 * X$^{7.23E+00}$ * e$^{-7.77E-02 * X}$

for X > 8

Signalized Arterials

Travel Time without Queuing (hours per vehicle mile)

$$H_u = 1 / \text{Speed} = (1 / Sf)\,(1 + 3.67E\text{-}02 * X^{8.58E\text{-}01})$$

Daily Delay Due to Recurring Queues (hours per bottleneck)

$$H_r = \text{RECURRING DELAY} = 0$$

for X <= 8

$$H_r = \text{RECURRING DELAY} = 0.00E{+}00 * (X\ \ 8)^2 + 6.89E\text{-}05 * (X\ \ 8)^3$$

for X > 8

Delay Due to Incidents for Arterials (hour per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * 1.17E\text{-}05 * X^{8.72E\text{-}01} * e^{1.75E\text{-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * 1.26E\text{-}05 * X^{8.12E\text{-}01} * e^{1.81E\text{-}01 * X}$$

for X > 8

Table 2.2 Key for Curve Fitting Results for Freeways and Signalized Arterials
A.M. Peak Direction, Peak Period Traffic

Freeways

Travel Time without Queuing (hours per vehicle mile)

$$H_u = 1 / \text{Speed} = (1 / Sf)\,(1 + 9.26E\text{-}12 * X^{10})$$

for X <= 8

$$H_u = 1 / \text{Speed} = (1 / Sf)\,(1.33E{+}00\ \ 1.05E\text{-}01 * X + 1.03E\text{-}02 * X^2\ \ 2.84E\text{-}04 * X^3)$$

for X > 8

Delay Due to Recurring Queues (hours per bottleneck)

$$H_r = \text{RECURRING DELAY} = 0$$

for X <=8

$$H_r = \text{RECURRING DELAY} = 7.52E\text{-}03 * (X\ \ 8)\ \ 2.12E\text{-}03 * (X\ \ 8)^2 + 1.07E\text{-}03 * (X\ \ 8)^3$$

for X > 8

Delay Due to Incidents for Two Lane Facilities (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 4.22 * (1\text{-}SF)^{1.05}) * 1.21E\text{-}07 * X^{5.07E+00} * e^{-1.96E\text{-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 4.22 * (1\text{-}SF)^{1.05}) * 3.04E\text{-}09 * X^{6.94E+00} * e^{-2.21E\text{-}01 * X}$$

for X > 8

Delay Due to Incidents for Three Lane Facilities (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.77 * (1\text{-}SF)^{1.04})$$

$$* 1.04E\text{-}05 * X^{-2.38E+00} * e^{1.14E+00 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SF})^{1.04})$$
$$* 7.39E\text{-}11 * X^{9.17E+00} * e^{-3.85E-01 * X}$$

for X > 8

Delay Due to Incidents for Four Lane Facilities (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04})$$
$$* 2.87E\text{-}07 * X^{1.59E-01} * e^{9.15E-01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04})$$
$$* 1.19E\text{-}11 * X^{1.04E+01} * e^{-4.73E-01 * X}$$

for X > 8

Delay Due to Accidents for Two Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 4.22 * (1\text{-SF})^{1.05})$$
$$* 4.12E\text{-}06 * X^{1.77E+00} * E^{1.71E-01 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 4.22 * (1\text{-SF})^{1.05})$$
$$* 8.24E\text{-}09 * X^{6.07E+00} * e^{-1.70E-01 * X}$$

for X > 8

Delay Due to Accidents for Three Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SF})^{1.04})$$
$$* 2.05E\text{-}08 * X^{5.94E+00} * e^{-3.09E-01 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SF})^{1.04})$$
$$* 9.73E\text{-}10 * X^{6.74E+00} * e^{-1.35E-01 * X}$$

for X > 8

Delay Due to Accidents for Four Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04})$$
$$* 6.06E\text{-}11 * X^{1.13E+01} * e^{-9.78E-01 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04})$$
$$* 2.08E\text{-}10 * X^{7.58E+00} * e^{-1.77E-01 * X}$$

for X > 8

Signalized Arterials

Travel Time without Queuing (hours per vehicle mile)

$$H_u = 1 / \text{Speed} = (1 / \text{Sf}) (1 + 3.25E\text{-}02 * X^{9.53E-01})$$

Daily Delay Due to Recurring Queues (hours per bottleneck)

$$H_r = \text{RECURRING DELAY} = 0$$

for X <=8

$$H_r = \text{RECURRING DELAY} = 2.87E\text{-}04 * (X \;\; 8)^{2} + 7.81E\text{-}05 * (X \;\; 8)^{3}$$

for X > 8

Delay Due to Incidents for Arterials (hours per vehicle mile)

H_i = INCIDENT DELAY = IncRate * $DurFac^2$ * 7.67E-06 * $X^{1.30E+00}$ * $e^{1.23E-01 * X}$

for X <= 8

H_i = INCIDENT DELAY = IncRate * $DurFac^2$ * 3.64E-06 * $X^{1.72E+00}$ * $e^{1.06E-01 * X}$

for X > 8

Table 2.3 Key for Curve Fitting Results for Freeways and Signalized Arterials
P.M. Peak Direction, Daily Traffic

Freeways

Travel Time without Queuing (hours per vehicle mile)

H_u = 1 / Speed = (1 / Sf) (1 + 7.37E-12* X^{10})

for X <= 8

H_u = 1 / Speed = (1 / Sf) (1.13E+00 4.39E-02* X + 4.68E-03 * X^2 1.32E-04 * X^3)

for X > 8

Delay Due to Recurring Queues (hours per bottleneck)

H_r = RECURRING DELAY = 0

for X <=8

H_r = RECURRING DELAY = 4.11E-03 * (X 8) + 1.26E-03 * $(X\ \ 8)^2$ + 4.03E-04 * $(X\ \ 8)^3$

Delay Due to Incidents for Two Lane Facilities (hours per vehicle mile)

H_i = INCIDENT DELAY = IncRate * $DurFac^2$ * (1 + 4.22 * $(1-SF)^{1.05}$)
* 2.45E-08 * $X^{6.90E+00}$ * $e^{-4.80E-01 * X}$

for X <= 8

H_i = INCIDENT DELAY = IncRate * $DurFac^2$ * (1 + 4.22 * $(1-SF)^{1.05}$)
* 4.38E-09 * $X^{6.68E+00}$ * $e^{-2.06E-01 * X}$

for X > 8

Delay Due to Incidents for Three Lane Facilities (hours per vehicle mile)

H_i = INCIDENT DELAY = IncRate * $DurFac^2$ * (1 + 3.77 * $(1-SF)^{1.04}$)
* 1.15E-07 * $X^{2.44E+00}$ * $e^{4.45E-01 * X}$

for X <= 8

H_i = INCIDENT DELAY = IncRate * $DurFac^2$ * (1 + 3.77 * $(1-SF)^{1.04}$)
* 1.33E-09 * $X^{7.21E+00}$ * $e^{-2.34E-01 * X}$

for X > 8

Delay Due to Incidents for Four Lane Facilities (hours per vehicle mile)

H_i = INCIDENT DELAY = IncRate * $DurFac^2$ * (1 + 3.45 * $(1-SF)^{1.04}$)
* 6.03E-09 * $X^{4.30E+00}$ * $e^{3.19E-01 * X}$

for X <= 8

H_i = INCIDENT DELAY = IncRate * $DurFac^2$ * (1 + 3.45 * $(1-SF)^{1.04}$)
* 3.86E-10 * $X^{7.92E+00}$ * $e^{-2.78E-01 * X}$

for X > 8

Delay Due to Accidents for Two Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 4.22 * (1\text{-SF})^{1.05}) * 4.12\text{E-}06 * X^{1.77\text{E+}00} * E^{1.71\text{E-}01 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 4.22 * (1\text{-SF})^{1.05}) * 8.24\text{E-}09 * X^{6.07\text{E+}00} * e^{-1.70\text{E-}01 * X}$$

for X > 8

Delay Due to Accidents for Three Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SF})^{1.04}) * 3.18\text{E-}07 * X^{1.84\text{E+}00} * e^{4.33\text{E-}01 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SF})^{1.04}) * 3.85\text{E-}09 * X^{6.16\text{E+}00} * e^{-1.38\text{E-}01 * X}$$

for X > 8

Delay Due to Accidents for Four Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04}) * 5.28\text{E-}08 * X^{2.09\text{E+}00} * e^{5.84\text{E-}01 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04}) * 8.65\text{E-}10 * X^{7.10\text{E+}00} * e^{-2.03\text{E-}01 * X}$$

for X > 8

Signalized Arterials

Travel Time without Queuing (hours per vehicle mile)

$$H_u = 1 / \text{Speed} = (1 / \text{Sf})\ (1 + 3.67\text{E-}02 * X^{8.58\text{E-}01})$$

Daily Delay Due to Recurring Queues (hours per bottleneck)

$$H_r = \text{RECURRING DELAY} = 0$$

for X <=8

$$H_r = \text{RECURRING DELAY} = 1.31\text{E-}03 * (X\ \ 8)^2 + 1.31\text{E-}03 * (X\ \ 8)^3$$

for X > 8

Delay Due to Incidents for Arterials (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * 4.19\text{E-}07 * X^{5.06\text{E+}00} * e^{-4.75\text{E-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * 4.98\text{E-}07 * X^{3.27\text{E+}00} * e^{-3.06\text{E-}02 * X}$$

for X > 8

Table 2.4 Key for Curve Fitting Results for Freeways and Signalized Arterials
P.M. Peak Direction, Peak Period Traffic

Freeways

Travel Time without Queuing (hours per vehicle mile)

$$H_u = 1 / \text{Speed} = (1 / Sf)\,(1 + 1.42E\text{-}11 * X^{10})$$

for X <= 8

$$H_u = 1 / \text{Speed} = (1 / Sf)\,(1.13E{+}00 \quad 5.19E\text{-}02 * X + 6.21E\text{-}03 * X^2 + \quad 1.85E\text{-}04 * X^3)$$

for X > 8

Delay Due to Recurring Queues (hours per bottleneck)

$$H_r = \text{RECURRING DELAY} = 0$$

for X <=8

$$H_r = \text{RECURRING DELAY} = 3.63E\text{-}03 * (X \quad 8) + 5.03E\text{-}03 * (X \quad 8)^2 + \quad 3.99E\text{-}05 * (X \quad 8)^3$$

Delay Due to Incidents for Two Lane Facilities (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 4.22 * (1\text{-}SF)^{1.05}) * 1.42E\text{-}11 * X^{7.42E+00} * e^{-5.31E\text{-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 4.22 * (1\text{-}SF)^{1.05}) * 1.52E\text{-}08 * X^{6.59E+00} * e^{-2.86E\text{-}01 * X}$$

for X > 8

Delay Due to Incidents for Three Lane Facilities (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.77 * (1\text{-}SF)^{1.04}) * 1.80E\text{-}08 * X^{4.96E+00} * e^{8.67E\text{-}02 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.77 * (1\text{-}SF)^{1.04}) * 2.97E\text{-}09 * X^{7.53E+00} * e^{-3.55E\text{-}01 * X}$$

for X > 8

Delay Due to Incidents for Four Lane Facilities (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.45 * (1\text{-}SF)^{1.04}) * 1.61E\text{-}09 * X^{6.48E+00} * e^{-1.48E\text{-}02 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.45 * (1\text{-}SF)^{1.04}) * 6.77E\text{-}10 * X^{8.46E+00} * e^{-4.22E\text{-}01 * X}$$

for X > 8

Delay Due to Accidents for Two Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 4.22 * (1\text{-}SF)^{1.05}) * 3.51E\text{-}06 * X^{1.51E+00} * E^{2.13E\text{-}01 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 4.22 * (1\text{-}SF)^{1.05}) * 3.55E\text{-}08 * X^{5.10E+00} * e^{-1.46E\text{-}01 * X}$$

for X > 8

Delay Due to Accidents for Three Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SF})^{1.04}) * 1.64\text{E-}07 * X^{2.44\text{E+}00} * e^{3.80\text{E-}01 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SF})^{1.04}) * 1.30\text{E-}08 * X^{5.77\text{E+}00} * e^{-1.70\text{E-}01 * X}$$

for X > 8

Delay Due to Accidents for Four Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04}) * 1.77\text{E-}09 * X^{6.19\text{E+}00} * e^{-3.70\text{E-}02 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04}) * 2.43\text{E-}09 * X^{6.85\text{E+}00} * e^{-2.50\text{E-}01 * X}$$

for X > 8

Signalized Arterials

Travel Time without Queuing (hours per vehicle mile)

$$H_u = 1 / \text{Speed} = (1 / \text{Sf})\ (1 + 4.58\text{E-}02 * X^{8.23\text{E-}01})$$

Daily Delay Due to Recurring Queues (hours per bottleneck)

$$H_r = \text{RECURRING DELAY} = 0$$

for X <=8

$$H_r = \text{RECURRING DELAY} = 2.69\text{E-}03 * (X\ \ 8)^2\ \ 1.23\text{E-}04 * (X\ \ 8)^3$$

for X > 8

Delay Due to Incidents for Arterials (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * 8.79\text{E-}07 * X^{4.18\text{E+}00} * e^{-3.18\text{E-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * 5.05\text{E-}07 * X^{3.56\text{E+}00} * e^{-8.69\text{E-}02 * X}$$

for X > 8

Table 2.5 Key for Curve Fitting Results for Freeways and Signalized Arterials
Both Directions, Daily Traffic, Accident Delay Equations Only

Freeways

Travel Time factor without Queuing, Hu (hour per vehicle mile)

$$H_u = 1 / \text{Speed} = (1 / S_{ff})\ (1 + 4.87\text{E-}12 * X^{10})$$

for X <= 8

$$H_u = 1 / \text{Speed} = (1 / S_{ff})\ (1.16\ \ 5.04\text{E-}02 * X + 4.88\text{E-}03\ X^2\ \ 1.30\text{E-}04\ X^{3})$$

for X > 8

Delay Due to Recurring Queues, H_r (hours per vehicle per bottleneck)

$$H_r = \text{RECURRING DELAY} = 0$$

for X<= 8

$$H_r = \text{RECURRING DELAY} = 4.69E\text{-}03 * (X\ \ 8)\quad 1.50E\text{-}03 * (X\ \ 8)^2 + 6.99E\text{-}04 * (X\ \ 8)^3$$

for X > 8

Delay Due to Incidents for Two Lane Freeways (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 4.22 * (1\text{-SFac})^{1.05}) * 3.98E\text{-}06 * X^{4.39E\text{-}01} * e^{5.32E\text{-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 4.22 * (1\text{-SFac})^{1.05}) * 1.89E\text{-}09 * X^{6.89} * e^{-1.89E\text{-}01 * X}$$

for X > 8

Delay Due to Incidents for Three Lane Freeways (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SFac})^{1.04}) * 1.21E\text{-}07 * X^{2.66} * e^{-3.27E\text{-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SFac})^{1.04}) * 2.46E\text{-}10 * X^{7.84} * e^{-2.44E\text{-}01 * X}$$

Delay Due to Incidents for Four Lane Freeways (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SFac})^{1.04}) * 2.51E\text{-}08 * X^{2.43} * e^{5.73E\text{-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SFac})^{1.04}) * 6.43E\text{-}11 * X^{8.63} * e^{-2.94E\text{-}01 * X}$$

for X > 8

Delay Due to Accidents for Two Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 4.22 * (1\text{-SF})^{1.05}) * 3.51E\text{-}06 * X^{1.51E+00} * e^{2.13E\text{-}01 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 4.22 * (1\text{-SF})^{1.05}) * 3.68E\text{-}09 * X^{6.16E+00} * e^{-1.38E\text{-}01 * X}$$

for X > 8

Delay Due to Accidents for Three Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SF})^{1.04}) * 1.13E\text{-}07 * X^{3.11E+00} * e^{1.81E\text{-}01 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SF})^{1.04}) * 1.42E\text{-}09 * X^{6.27E+00} * e^{-9.20E\text{-}02 * X}$$

for X > 8

for X > 8

Delay Due to Accidents for Four Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04}) * 1.23\text{E-}06 * X^{-1.49\text{E+}00} * e^{1.07\text{E+}00 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04}) * 3.36\text{E-}10 * X^{7.09\text{E+}00} * e^{-1.36\text{E-}01 * X}$$

for X > 8

Signalized Arterials

Travel Time without Queuing (hours per vehicle mile)

$$H_u = 1 / \text{Speed} = (1 / S_{ff})\ 1 + 0.0337 * X^{0.856})$$

Delay Due to Recurring Queues (hours per vehicle per bottleneck)

$$H_r = \text{RECURRING DELAY} = 0$$

for X <= 8

$$H_r = \text{RECURRING DELAY} = 3.08\text{E-}04 * (X\ \ 8)^2 + 2.26\text{E-}05 * (X\ \ 8)^3$$

for X > 8

Delay Due to Incidents for Arterials (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2\ 1.69\text{E-}05 * X^{1.17\text{E-}01} * e^{3.33\text{E-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2\ 2.98\text{E-}06 * X^{1.94} * e^{7.63\text{E-}02 * X}$$

for X > 8

Delay Due to Accidents for Four Lane Facilities (hours per vehicle mile)

$$\text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04}) * 1.23\text{E-}06 * X^{-1.49\text{E+}00} * e^{1.07\text{E+}00 * X}$$

for X <= 8

$$\text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04}) * 3.36\text{E-}10 * X^{7.09\text{E+}00} * e^{-1.36\text{E-}01 * X}$$

for X > 8

Table 2.6 Key for Curve Fitting Results for Freeways and Signalized Arterials
Both Directions, Peak Period Traffic

Freeways

Period Delay Due to Recurring Queues (hours per vehicle per bottleneck)

$$H_r\ \text{RECURRING DELAY} = 0$$

for X <= 8

$$H_r = \text{RECURRING DELAY} = 5.56\text{E-}03 * (X\ \ 8) + 1.44\text{E-}03 * (X\ \ 8)^2 + 5.17\text{E-}04 * (X\ \ 8)^3$$

for X > 8

Period Delay Due to Incidents for Two Lane Freeways (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 4.22 * (1\text{-SFac})^{1.05}) * 4.94\text{E-}06 * X^{3.16\text{E-}01} * e^{6.16\text{E-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 4.22 * (1\text{-SFac})^{1.05}) * 8.79\text{E-}09 * X^{6.68} * e^{-2.46\text{E-}01 * X}$$

for X > 8

Period Delay Due to Incidents for Three Lane Freeways (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SFac})^{1.04}) * 5.23\text{E-}07 * X^{1.06} * e^{6.53\text{E-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SFac})^{1.04}) * 6.87\text{E-}10 * X^{8.13} * e^{-3.56\text{E-}01 * X}$$

for X > 8

Period Delay Due to Incidents for Four Lane Freeways (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SFac})^{1.04}) * 1.99\text{E-}08 * X^{3.45} * e^{4.31\text{E-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SFac})^{1.04}) * 1.41\text{E-}10 * X^{9.14} * e^{-4.31\text{E-}01 * X}$$

for X > 8

Delay Due to Accidents for Two Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 4.22 * (1\text{-SF})^{1.05}) * 3.51\text{E-}06 * X^{1.51\text{E+}00} * E^{2.13\text{E-}01 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 4.22 * (1\text{-SF})^{1.05}) * 3.68\text{E-}09 * X^{6.16\text{E+}00} * e^{-1.38\text{E-}01 * X}$$

for X > 8

Delay Due to Accidents for Three Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SF})^{1.04}) * 1.13\text{E-}07 * X^{3.11\text{E+}00} * e^{1.81\text{E-}01 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.77 * (1\text{-SF})^{1.04}) * 1.42\text{E-}09 * X^{6.27\text{E+}00} * e^{-9.20\text{E-}02 * X}$$

for X > 8

Delay Due to Accidents for Four Lane Facilities (hours per vehicle mile)

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04}) * 1.23\text{E-}06 * X^{-1.49\text{E+}00} * e^{1.07\text{E+}00 * X}$$

for X <= 8

$$H_a = \text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04}) * 3.36\text{E-}10 * X^{7.09\text{E+}00} * e^{-1.36\text{E-}01 * X}$$

for X > 8

Signalized Arterials

Period Time without Queuing (hours per vehicle mile)

$$H_u = 1 / \text{Speed} = (1 / S_{ff})\ (1 + 0.0392 * X^{0.869})$$

Period Delay Due to Recurring Queues (hours per vehicle per bottleneck)

$$H_r = \text{RECURRING DELAY} = 0$$

for X <= 8

$$H_r = \text{RECURRING DELAY} = 9.87\text{E-}04 * (X\ \ 8)^2 \quad 1.42\text{E-}05 * (X\ \ 8)^3$$

for X > 8

Period Delay Due to Incidents for Arterials (hours per vehicle mile)

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * 1.16\text{E-}05 * X^{8.08\text{E-}01} * e^{2.23\text{E-}01 * X}$$

for X <= 8

$$H_i = \text{INCIDENT DELAY} = \text{IncRate} * \text{DurFac}^2 * 1.35\text{E-}06 * X^{2.69} * e^{3.69\text{E-}03 * X}$$

for X > 8

Delay Due to Accidents for Four Lane Facilities (hours per vehicle mile)

$$\text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04}) * 1.23\text{E-}06 * X^{-1.49\text{E+}00} * e^{1.07\text{E+}00 * X}$$

for X <= 8

$$\text{ACCIDENT DELAY} = (\text{AccRate} - 1) * \text{DurFac}^2 * (1 + 3.45 * (1\text{-SF})^{1.04}) * 3.36\text{E-}10 * X^{7.09\text{E+}00} * e^{-1.36\text{E-}01 * X}$$

for X > 8

Where:

X	=	AADT/C ratio (0 to 18);
S_{ff}	=	free flow speed;
IncRate	=	incident rate factor;
	=	(target incident rate)/(default incident rate);
DurFac	=	duration factor;
	=	(target mean incident duration)/38.0;
SFac	=	shoulder factor;
	=	1.0 for usable shoulders both sides;
	=	0.5 for usable shoulders one side only;
	=	0 for no usable shoulders;
AccRate	=	accident rate factor; and
	=	(target accident rate)/(default accident rate).

Note that the selection of the incident rate, accident rate, and duration factors are based on comparison to the default values. For incident duration, the overall weighted average duration of all incidents for the default case is 38.0 minutes. Because both accident rate varies with traffic volume, Table 2.7 was prepared as a guide for setting the defaults. Selected plots of these equations appear in Figures 2.1 to 2.6.

Figure 2.1 Incident Delay: Two-Lane Freeway

Incident Rate Factor

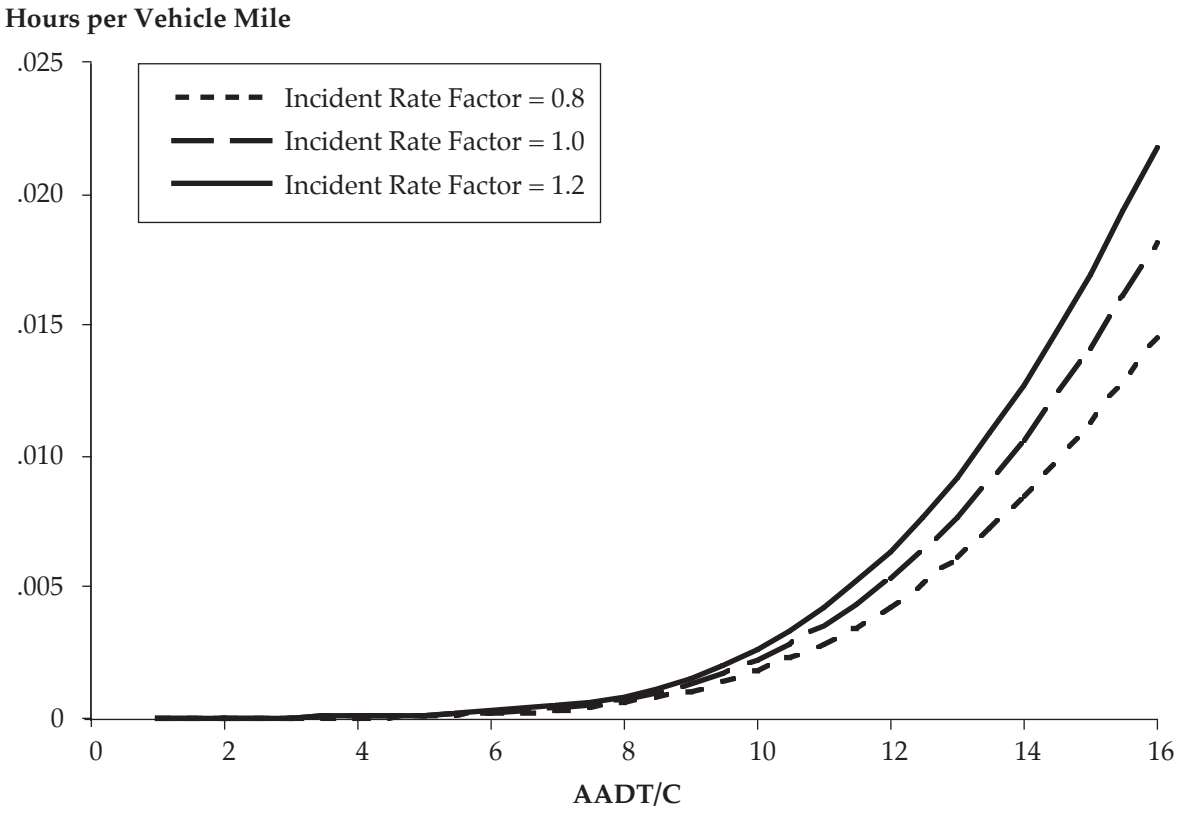


Figure 2.2 Incident Delay: Two-Lane Freeway

Duration Factor

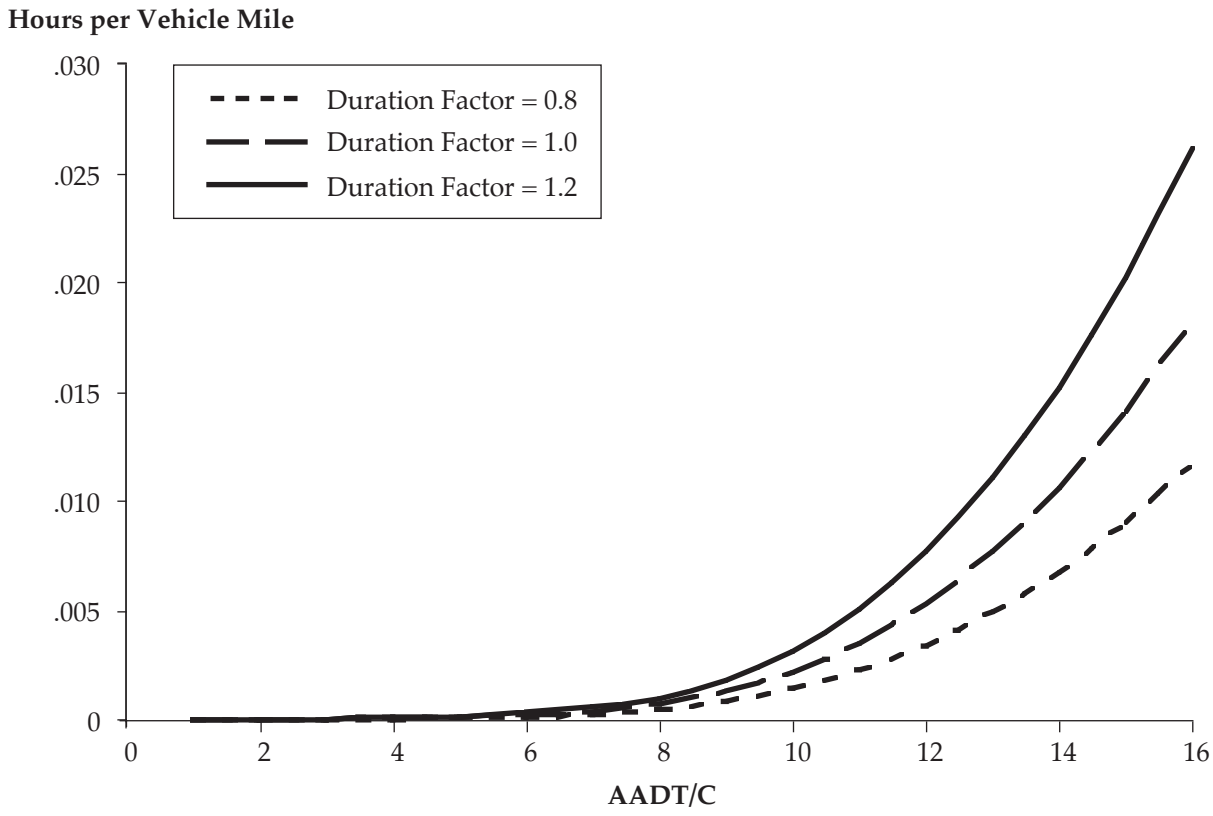


Figure 2.3 Incident Delay: Two-Lane Freeway

Shoulder Factor

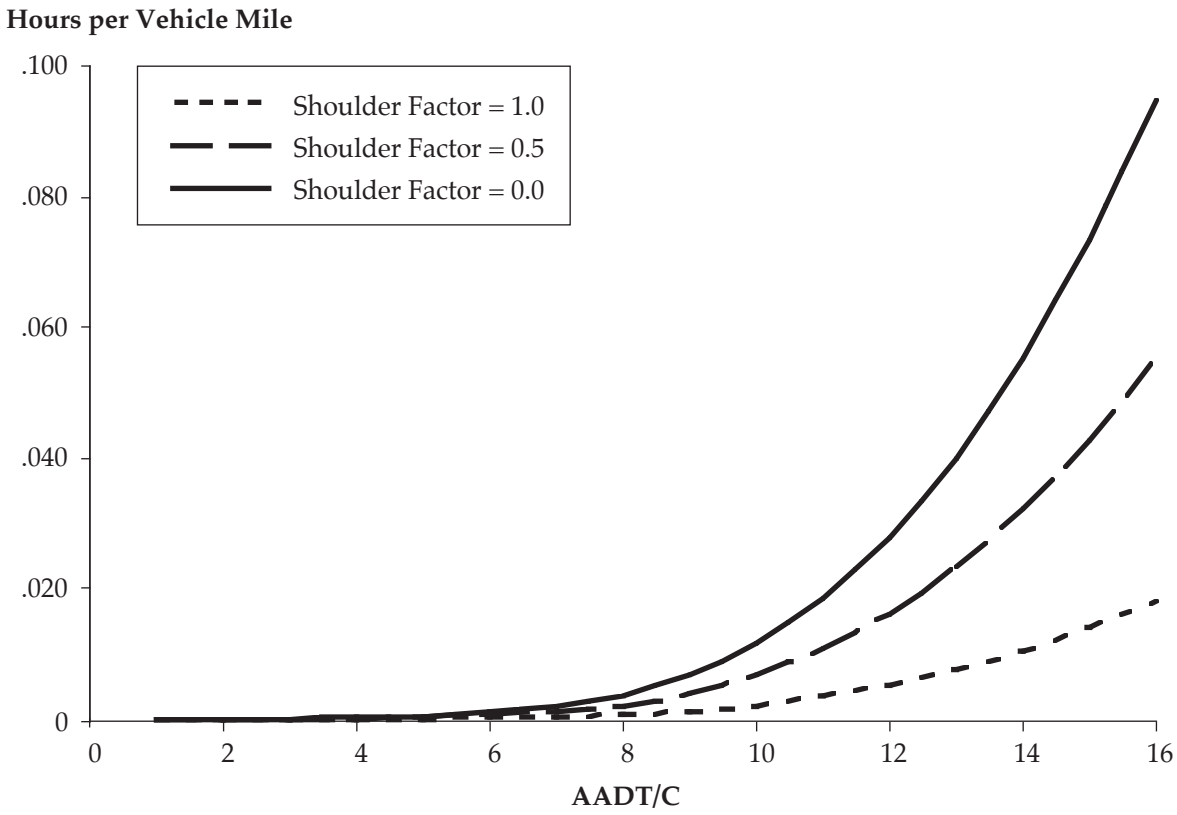


Figure 2.4 Freeway Incident Delay

Shoulder Factor = 0

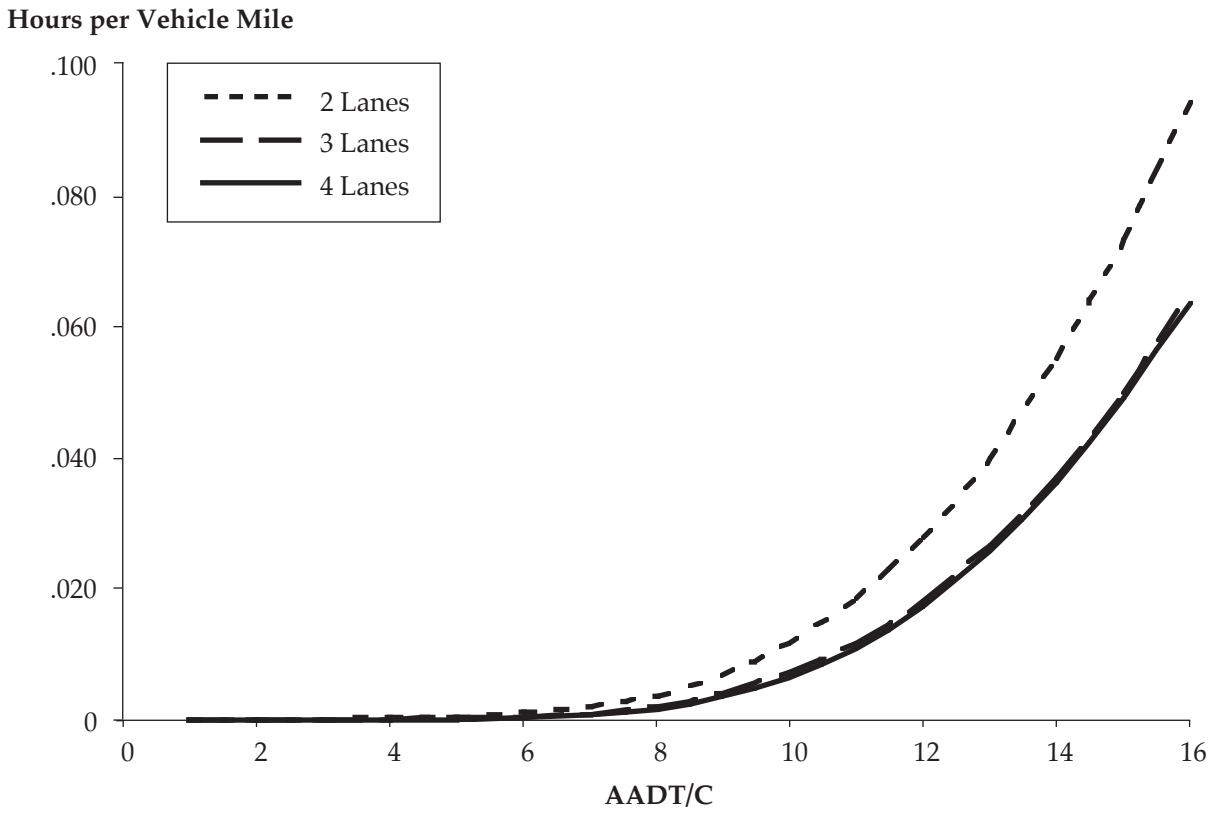


Figure 2.5 Freeway Incident Delay

Shoulder Factor = 0.5

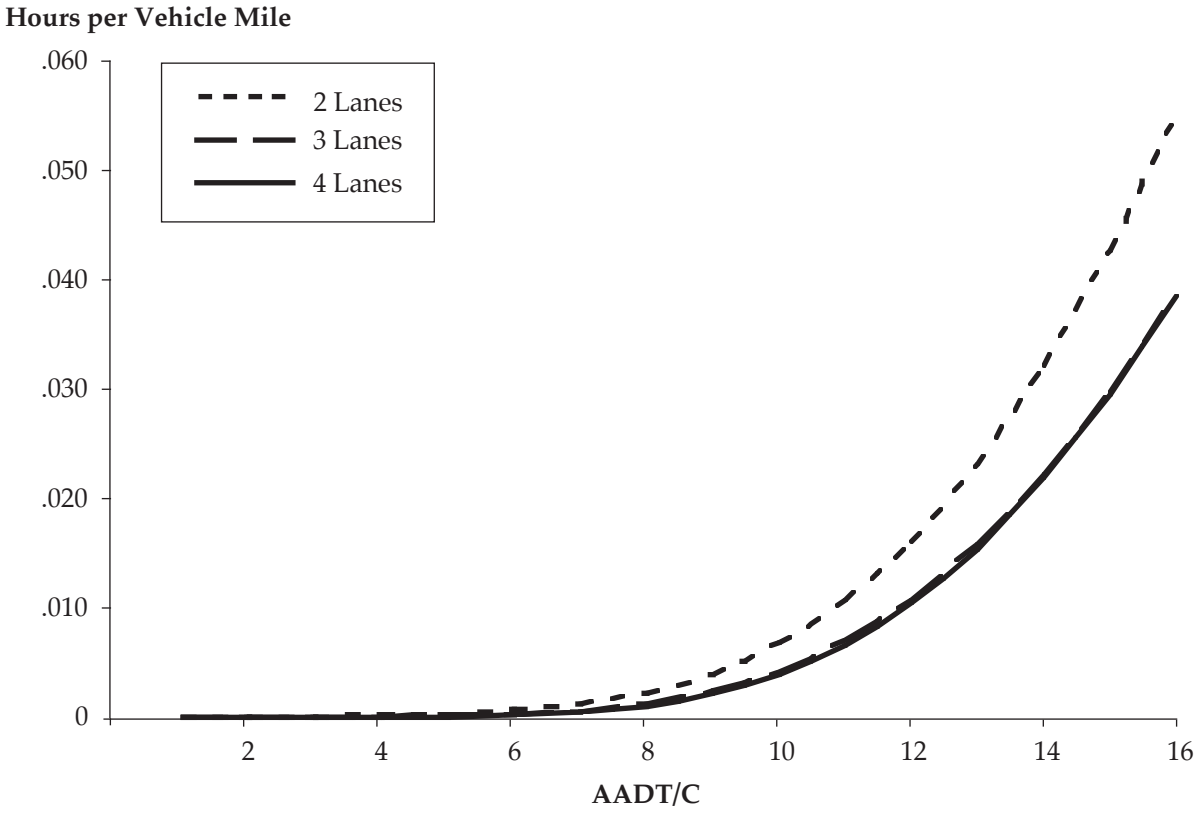


Figure 2.6 Freeway Incident Delay

Shoulder Factor = 1.0

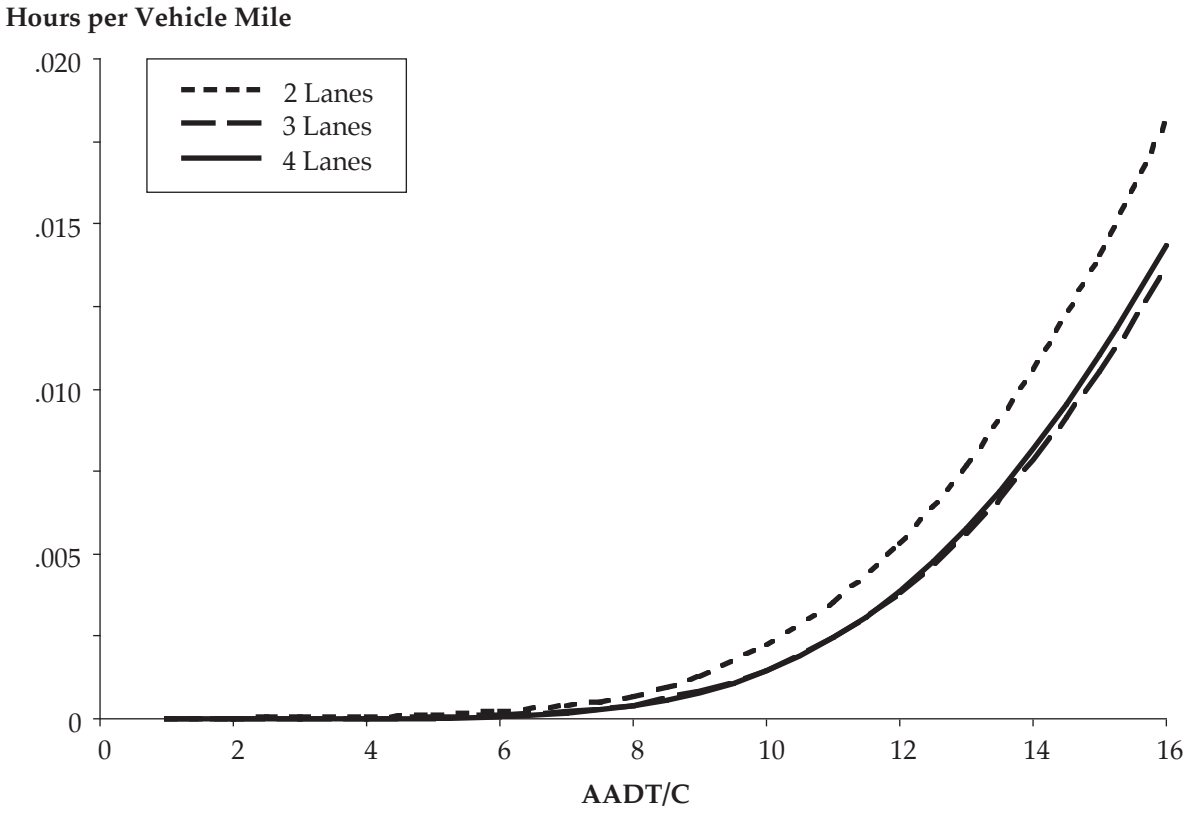


Table 2.7 Default Accident and Incident Rates by AADT/C

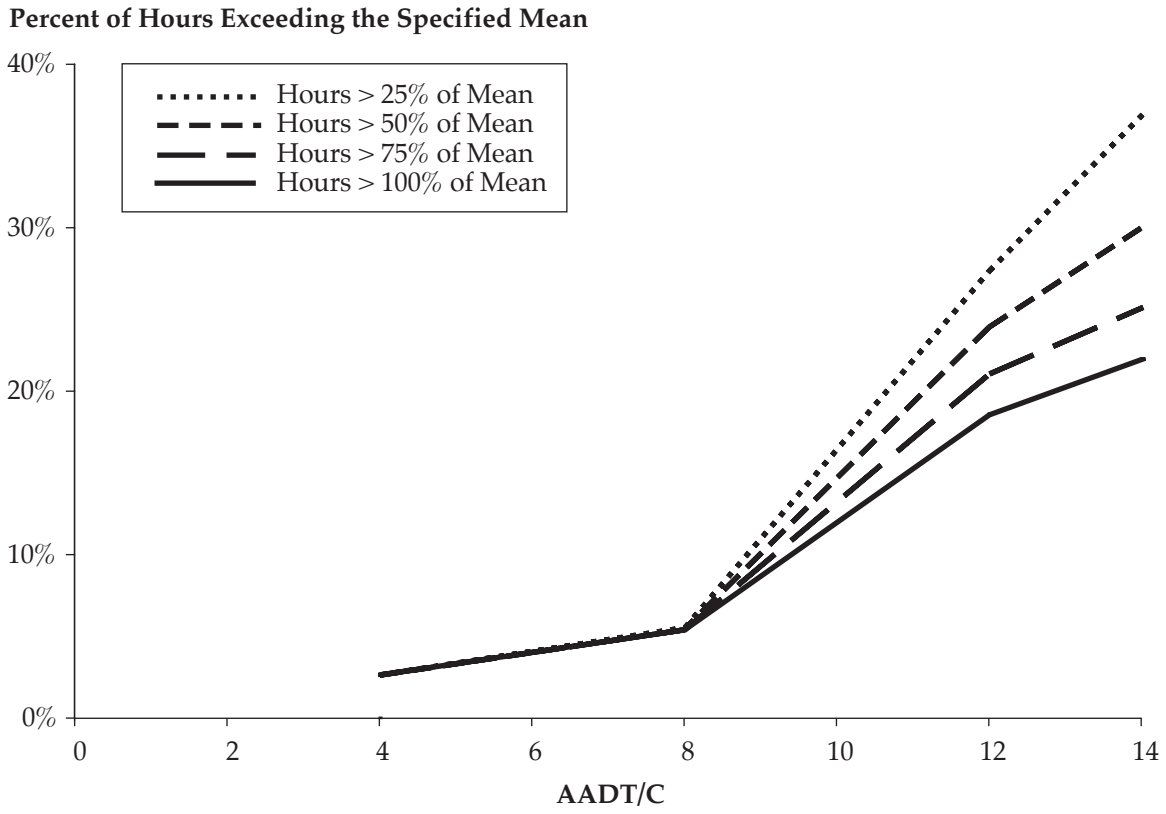
AADT/C	Accident Rate(per MVMT)	Total Incident Rate (per MVMT)
1	1.066	9.611
2	1.069	9.614
3	1.075	9.620
4	1.086	9.631
5	1.105	9.650
6	1.132	9.677
7	1.172	9.717
8	1.220	9.765
9	1.275	9.820
10	1.345	9.890
11	1.414	9.959
12	1.518	10.063
13	1.583	10.128
14	1.657	10.202
15	1.709	10.254
16	1.760	10.305
17	1.810	10.355
18	1.853	10.398

2.2 Variability in Delay Estimates

The stochastic nature of QSIM allows for computing the variability in delay estimates. Delay (travel time) variability is due to fluctuations in demand volume from day-to-day as well as to incident occurrence. Figures 2.7 and 2.8 display some aspects of delay variability for the base case scenario (all factors set to the default values). In these, the average delay *due to recurring congestion only* was first computed as a baseline. Then, an analysis of the percentage of hours in the peak period that exceed this average delay by 25, 50, 75 and 100 percent was performed. At AADT/C levels below 8.0, there is very little variability in recurring delay because volumes never get high enough for a queue to form (even allowing them to vary stochastically). What delay exists is theoretically due almost solely to incidents, and the variability is low less than five percent of hours in the peak period exceed the average by any amount. Since the lines are coincident at AADT/C values of 8.0 or less, it is clear that a small number of extreme incidents are causing the variability. Above an AADT/C of 8.0, variability increases dramatically due to the combined effect of incidents and recurring queues (which are caused by variations in volume). Figure 2.8 separates out the effects of incidents and recurring queues; the effect of incidents only can be found by subtracting values indicated by the two lines.

Figure 2.7 Peak Period Delay Variability

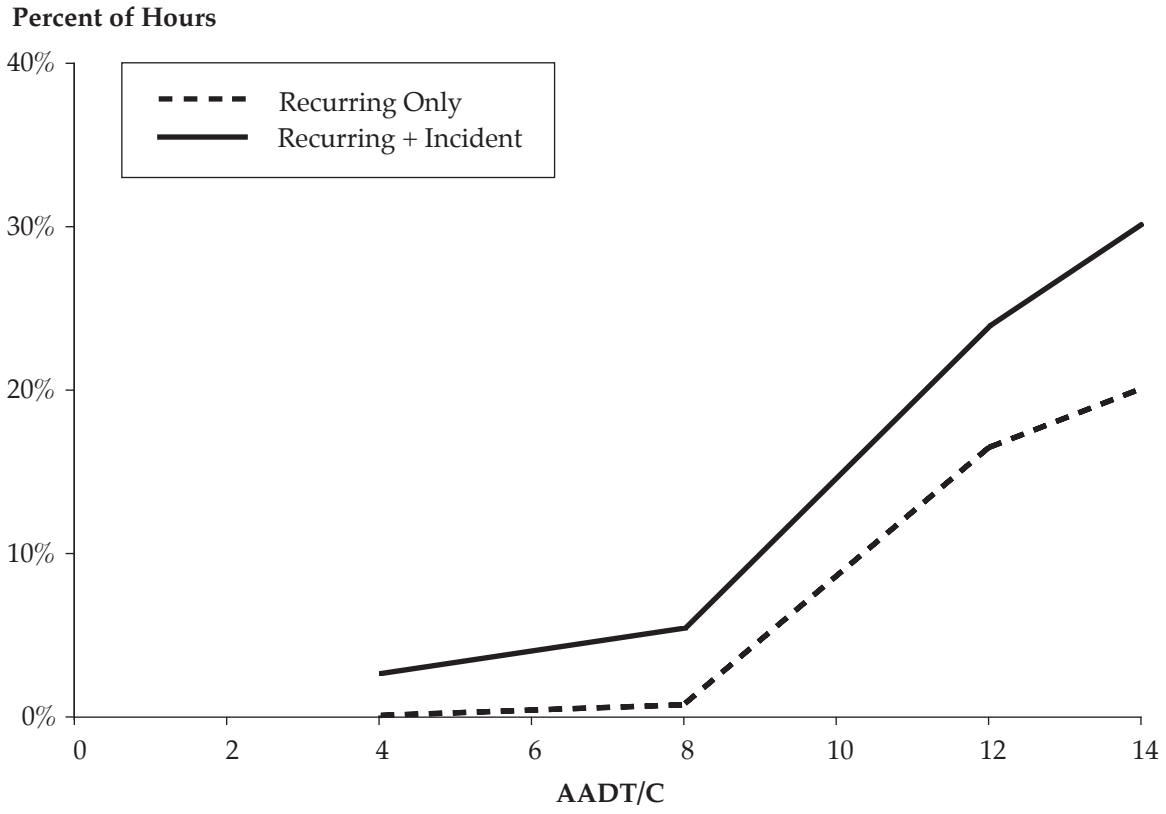
Combined Effect of Recurring Congestion and Incidents



Example: At AADT/C=12, 18% of hours in the peak period have delays that exceed the average delay by 100 or more percent.

Figure 2.8 Percent of Hours with Delay Greater than 50% of the Average

Peak Period



Thus, at an AADT/C of 8.0, it can be verified that almost all of the variability is attributable to incidents. At an AADT/C of 12.0, about 70 percent of peak-period variability is due to recurring congestion, an indication that while incidents of substantial impact do not happen every day while recurring congestion is more or less predictably high.

A few final notes on variability are in order. First, as can be seen in Figure 2.8, recurring congestion is subject to a good deal of variability within the model. This is because traffic volumes are allowed to fluctuate from day-to-day. Many other studies do not consider volume variations to be an aspect of recurring congestion, the assumption being that "recurring" congestion occurs in basically the same manner every day. This is probably not realistic since when volumes are near or above capacity, small changes affect delay in a nonlinear fashion. We have chosen to assign volume variability to recurring delay to distinguish it from incident-related delay.

Second, the model assumes that hourly traffic varies in accordance with data developed in Reference 3. That is, hourly coefficients of variation (CVs) are used to set the test volume for a given pass in the stochastic process. These data are composites developed from national data (more than 700 locations). CVs are important in determining not only variability but the average amount of delay because of the nonlinear nature of the volume/delay relationship. (High CVs will cause more occasions when volumes exceed capacity.) Individual facilities may exhibit less variability in their hourly volumes, particularly those that are heavily used by commuters. To test the viability of the default coefficients of variation, ITS surveillance data from Orlando (I-4), Seattle (I-5), and Denver (I-25) were examined. (More detail on these datasets are presented in the next Chapter). The Seattle data indicated slightly higher hourly CVs, the Denver CVs were roughly the same as the defaults, and the Orlando CVs were substantially lower than the defaults (almost half as large). All of these data represented only two-three months of data, so the actual annual CVs are likely to be larger. Therefore, as a national model the procedure used is reasonable. However, if conditions on a specific facility vary substantially from the defaults, absolute values for delay can be inaccurate. However, since the main purpose of the methodology is to gauge *relative* changes in delay due to implementing improvement strategies, it can still yield valuable information to planners.

Third, another way to consider variability is to look at the final corridor-wide estimates of recurring and incident delay predicted by the equations. These are based on average delay values. The predicted (average) incident delay can be thought of as additional delay over and above what can be expected from recurring bottlenecks. For example, consider a corridor that has been analyzed in accordance with the Application Guidelines and recurring delay is found to be 1,000 VHT and incident delay is found to be 500 VHT; these are the vehicle-hours of delay that can be expected on an "average" day. Therefore, on an average day, delay is increased by an additional 50 percent due to incidents. This increase could be thought of as variability in the base (recurring) delay estimates.

2.3 Estimation of Queue Lengths

Although the primary outputs from the modeling procedure are estimates of VHT and VMT, it is also possible to track queue lengths hour-by-hour. Currently, the model keeps

tracks of queues in terms of number of vehicles. Queue lengths in terms of distance are a complex function of vehicle spacing (headway) in queues which are in turn a function of bottleneck severity (as shown by the FRESIM experiments conducted for this study.) Future versions of the model can use these queue traffic parameters but for simplicity the current study focused strictly on number of vehicles in queues. Also, it is possible to fit equations to estimate queue lengths in the same manner as for delay, but this was not performed here. Rather, a cursory analysis of queues was performed. Figure 2.9 displays the longest *average* hourly queue for the peak period. Note that there is very little difference in the recurring- and incident-caused queues on average; this is because incidents occur only rarely and while recurring congestion happens regularly. In terms of the single longest queue experienced, Figure 2.10 shows the effect of very rare but extremely severe incidents; these cause single occurrences of delay that are several orders of magnitude higher than the longest delay caused by recurring conditions. The fact that the lines in Figure 2.10 are not entirely smooth are a reflection of the stochastic process even with 15,000 replicates at each AADT/C level some minor perturbations exist. For application, users will want to smooth these curves by hand to ensure consistent results.

Figure 2.9 Average Hourly Queue Length, Peak Period

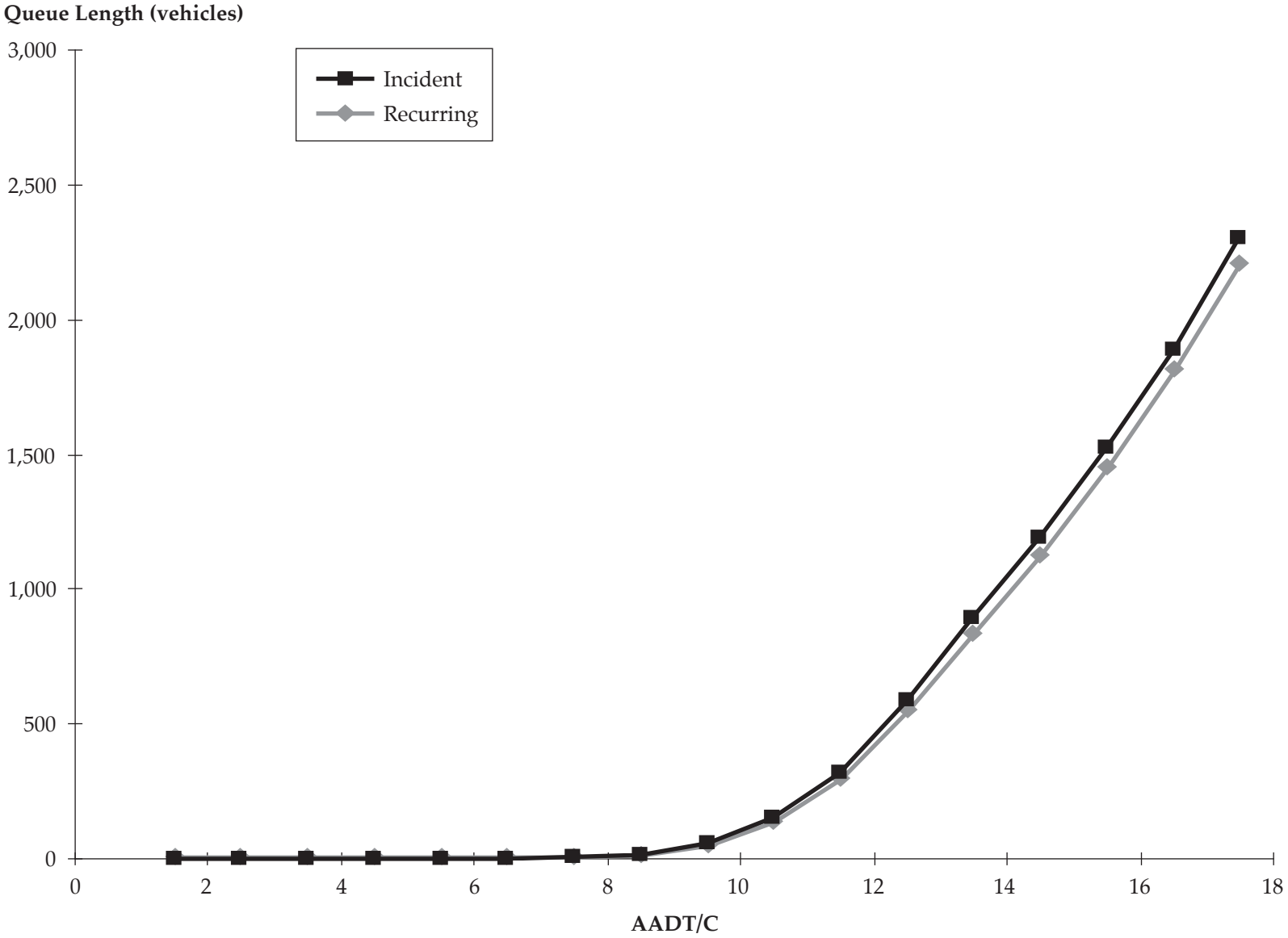
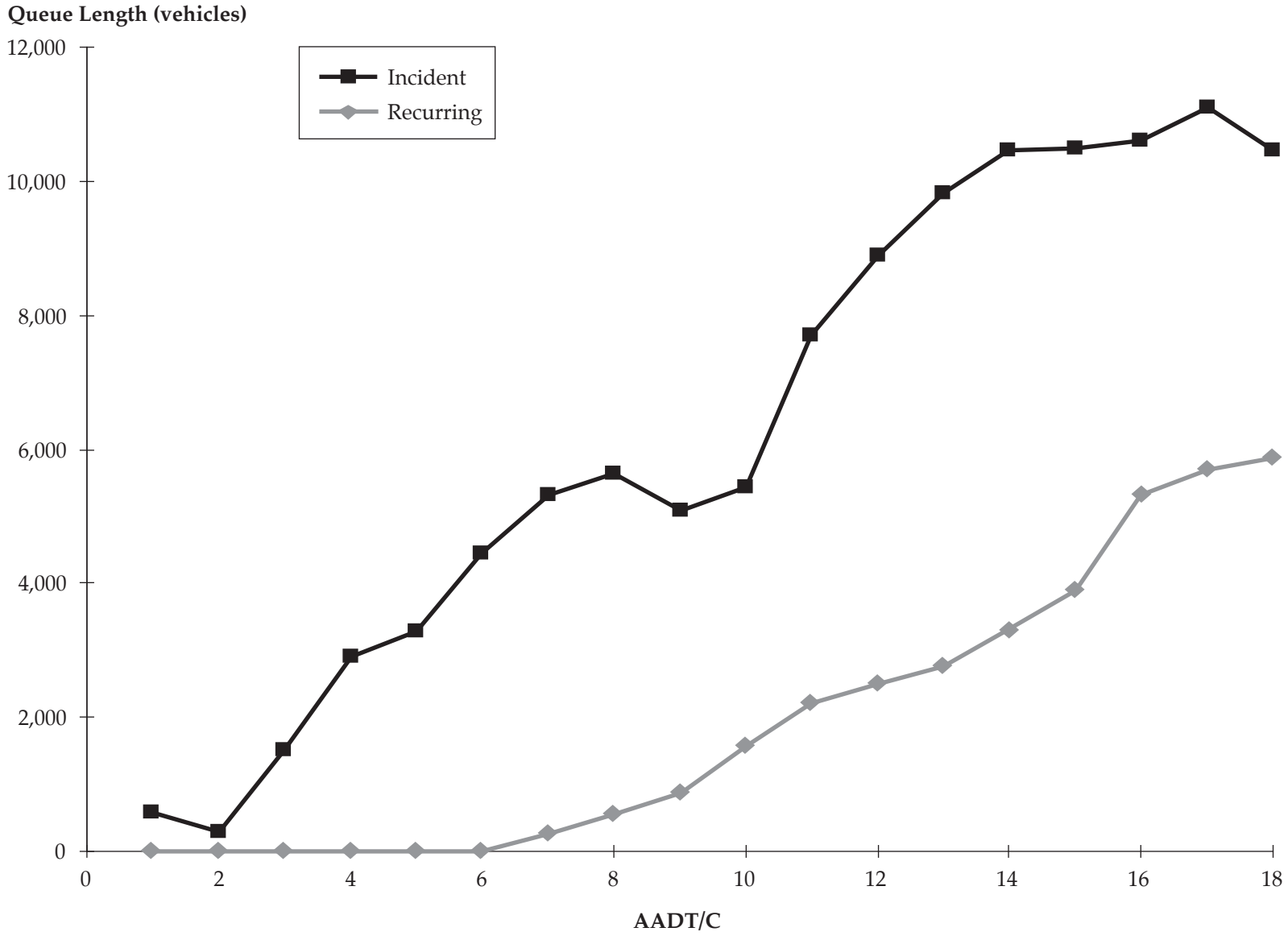


Figure 2.10 Longest Hourly Queue Lengths, Peak Period



3.0 Application Guidelines for Sketch Planning Incident Analysis

3.1 Freeways

Introduction

The sketch planning procedure for estimating incident impacts is meant to be applied for an extended section of freeway that can be anywhere from two to 20+ miles in length. The following input data and calculations are required.

Step 1: Decide If Directional Analysis is Needed

If traffic conditions vary substantially by direction, then each direction should be analyzed separately. This determination can be made based on directional volumes or the nature of bottlenecks (e.g., lane imbalance). For example, a lane-drop may exist in one direction and not the other, or an interchange may function as bottleneck in only one direction. Essentially, if capacity and volumes are close to being the same in both directions, then the corridor can be analyzed using combined directions; otherwise directional analysis should be undertaken.

Step 2: Identify Recurring Bottlenecks

The recurring bottlenecks in the corridor must be identified by the user. Based on the analysis undertaken with the QSIM model, recurring bottlenecks can occur whenever AADT/C values exceed 8.0. These can be on-ramps, freeway-to-freeway merges, lane-drops, or work zones. Because the procedure is highly sensitive to the specification of recurring bottlenecks in the corridor, care should be taken to identify "true" bottlenecks from areas where traffic breaks down due to downstream conditions. For example, the queue from a "true" bottleneck may spread upstream and cause traffic flow in another on-ramp area to breakdown. It is possible to have several successive segments with AADT/C ratios greater than 8.0, yet only designate one of them as the "true" or "controlling" recurring bottleneck. Under no circumstances should two successive links be coded as recurring bottlenecks; only one will control traffic flow.

Step 3: Define Links

The corridor should be broken into links for separate analysis. Because AADT/C is the basis for delay prediction, links should be defined when either AADT or capacity changes significantly. Logical breakpoints include on-ramps, lane-drops, freeway-to-freeway merges, and workzones.

Step 4: Set Input Parameters

1. **Analysis Period:** Select peak period or daily analysis periods.

2. **Free Flow Speed (FFS):** For freeways, free flow speed is the common definition in practice: the speed of a vehicle under very light traffic conditions. Following guidance from *NCHRP 387*[1], the following equations should be used:

$$FFS = (0.88 * SpeedLimit) + 14, \text{ for posted speed limits} > 50 \text{ mph} \quad (1)$$

$$FFS = (0.79 * SpeedLimit) + 12, \text{ for posted speed limits} <= 50 \text{ mph} \quad (2)$$

3. **Annual Average Daily Traffic (AADT):** The AADT of a section or link is the same as defined in the *Traffic Monitoring Guide*. If a travel demand forecasting model (TDF) model is used and if the model predicts weekday traffic, then adjustments must be made to TDF link volumes to correspond with the AADT definition. If peak hour forecasts are made, they first must be converted to Annual Average Weekday Traffic (AWDT) by factoring:

$$AAWT = \frac{PHV}{F_{PHV}} \quad (3)$$

where PHV is the forecasted peak-hour volume and F_{PHV} is the percent of daily traffic moving in the peak hour. If peak-hour forecasts are adjusted internally to correspond with design hour volumes (e.g., 30th highest annual hour of traffic), then F_{PHV} should be set to the local K-factor. If peak-hour forecasts represent the average "typical" peak hour, then F_{PHV} should be based on the average peak hour percentage of traffic as determined from local continuous count stations. If local values are unavailable, the following default percentages may be used:[2]

[1] *Planning Techniques to Estimate Speeds and Service Volumes for Planning Applications*, NCHRP Report 387, Transportation Research Board, 1997.

[2] *Development of Diurnal Traffic Distribution and Daily, Peak, and Off-peak Vehicle Speed Estimation Procedures for Air Quality Planning*, developed by COMSIS Corp. and SAIC for FHWA Office of Environment and Planning Work Order B-94-06, April 1996.

Highway Type	AADT/C Range	Default Peak-Hour Percent (One Direction)
Freeway	<= 7.5	0.0485
	7.5 < AADT/C <= 8.5	0.0469
	8.5 < AADT/C <= 9.5	0.0459
	9.5 < AADT/C <= 10.5	0.0438
	10.5 < AADT/C <= 11.5	0.0414
	11.5 < AADT/C <= 12.5	0.0390
	> 12.5	(see Equation 4)
Nonfreeway	<= 7.5	0.0483
	7.5 < AADT/C <= 8.5	0.0466
	8.5 < AADT/C <= 9.5	0.0455
	9.5 < AADT/C <= 10.5	0.0436
	10.5 < AADT/C <= 11.5	0.0414
	11.5 < AADT/C <= 12.5	0.0392
	> 12.5	(see Equation 4)

$$\text{Peak hr pct} = \frac{(0.0392 * (24 - AADT/C)) + ((1/48) * (AADT/C - 12))}{12} \quad (4)$$

For TDF models, if volumes are based on one-way links, then these links must be combined to represent two-way flow, i.e., the one-way AAWTs should be added.

Once two-way AAWT is obtained, it must be converted to AADT by dividing by the ratio of AAWT/AADT:

$$AADT = \frac{AAWT}{F_{AWDT}} \quad (5)$$

where: F_{AWDT} is the area-wide ratio of AAWT/AADT.

The default value for F_{AWDT} is 1.0757.

If directional analysis is chosen, AADT should still be computed on the basis of both directions combined (to be consistent with how the equations were developed). Thus, if one direction of a freeway has an AADT value of 50,000 vehicles per day (vpd) and the other has an AADT of 60,000, the AADTs to use in the equations are 100,000 and 120,000, respectively.

4. **Capacity:** The capacity for all **through/general purpose lanes only** should be calculated. Do not consider extended acceleration lanes and lanes added to improve the functioning of interchange areas. If the area is a workzone, then capacity needs to account for the nature of the work zone. For all facilities, capacity is based on the *Highway Capacity Manual (HCM)* capacity for the peak hour. Freeway capacity for the speed/delay models is the same as defined by the *HCM*: the maximum sustainable flow rate past a point on the highway (i.e., flow rate at Level of Service E). All attempts should be made to follow Chapter 3 of the *HCM* in computing this value. If

it is not available, then the following calculations, based on Equation 11-1 from *NCHRP 387* can be made.

$$\text{Capacity (vph)} = \text{IdealCap} * N * FHV * PHF \quad (6)$$

where:

IdealCap	=	2,400 pcphpl if free flow speed >= 70 mph; 2,300 otherwise
N	=	number of through lanes
F_{HV}	=	heavy vehicle adjustment factor
	=	1.0/(1.0 + 0.5 HV) for level terrain
	=	1.0/(1.0 + 2.0 HV) for rolling terrain
	=	1.0/(1.0 + 5.0 HV) for mountainous terrain (rare in urban areas)
HV	=	daily proportion of trucks and busses in traffic stream
PHF	=	ratio of peak 15-min flow rate to average hourly rate

The PHF factor should be chosen with care because the default value recommended by *NCHRP 387* (0.90) can have a significant influence on capacity. Examination of freeway surveillance data from Orlando (I-4) and Denver (I-25) show that for these urban interstates the PHF is approximately 0.95. Further, if local evidence exists that the ideal capacities given above can be sustained for hour-long periods, then the PHF should be set to 1.0.

HV should be computed as a composite average for the corridor to avoid having capacity vary from section to section.

In the predictive equations, capacity is the sum of the one-way capacities. Thus, assuming the result of Equation 6 is 2,100 vph and there are three lanes in each direction, capacity in the AADT/C term of the equations is (6 x 2,100 = 12,600). If directional analysis is chosen capacity should still be computed on the basis of both directions combined. Continuing the example, if the freeway segment has three lanes is one direction and two in the other, then capacities in the AADT/C term are 6,300 and 4,200, respectively.

5. **AADT/C:** For base/current year conditions, if AADT/C exceeds 13.0, then both AADT and capacity should be checked for accuracy. (AADT/C rarely exceeds 14.0 for existing facilities.) Under no conditions, including forecasted future volumes, should AADT/C exceed 18.0.

6. **Incident Rate Factor:** If the incident rate of the facility is known, it should be used in the equations. It is computed as:

$$\text{IncRateFac} = (\text{Facility Incident Rate})/(\text{Default Incident Rate}) \quad (7)$$

Here, the incident rate includes all forms of incidents, even minor ones. The types considered in the default model are: 1) abandoned vehicles, 2) accidents (crashes), 3) debris on roadway, 4) vehicle breakdowns (mechanical trouble, stalled vehicles, flat tires), and 5) "other" (vehicles parked without having a breakdown). If information on **all** of these types are not available, then the analyst should either use the default incident rate or can

factor their data using the distribution of incident types information provided in Chapter 3.

Note that the selection of the Incident Rate, Accident Rate, and Duration Factors (see below) are based on comparison to the default values. (If these are not known for the facility being analyzed, they should be set to 1.0.) Because individual links on the facility will usually have different AADT/C ratios, the Incident Rate Factor should be developed for each individual link (Table 3.1 offers guidance.)

Table 3.1 Default Accident and Incident Rates by AADT/C

AADT/C	Accident Rate (per MVMT)	Total Incident Rate (per MVMT)
1	1.066	9.611
2	1.069	9.614
3	1.075	9.620
4	1.086	9.631
5	1.105	9.650
6	1.132	9.677
7	1.172	9.717
8	1.220	9.765
9	1.275	9.820
10	1.345	9.890
11	1.414	9.959
12	1.518	10.063
13	1.583	10.128
14	1.657	10.202
15	1.709	10.254
16	1.760	10.305
17	1.810	10.355
18	1.853	10.398

7. **Accident Rate Factor:** Accidents are a subset of total incidents. As with the incident rate factor, if the accident rate of the facility is known, it should be used in the equations. It is computed as:

$$\text{AccRateFac} = (\text{Facility Accident Rate}/\text{Default Accident Rate}) - 1.0 \quad (8)$$

The Accident Rate Factor is developed differently than the other factors. It essentially measures the deviation of the facility-specific rate from the default accident rate that is imbedded in the overall incident rate. In other words, it is used to adjust the calculated incident delay to account for accident rates that are higher or lower than the default. Therefore, the results of the equation for accidents only should be added to the results of the equation for incidents (see Step 6 below).

Since accident rate varies by traffic volume it should be computed for each link based on its AADT/C level and the values in Table 3.1. *To avoid double counting, either the total incident rate* ***or*** *the accident rate should be adjusted for accidents, but not both.* For example, if the accident rate of a facility is known but not the incident rate, the default incident rate should be used and the accident rate factor adjusted as shown in Equation 8. If both are known, it is recommended that the incident rate factor only be adjusted.

8. **Duration Factor:** For incident duration, the overall weighted average duration of all incidents for the default case is 38.0 minutes. The duration factor is then:

$$\text{DurFac} = (\text{target mean incident duration})/38.0 \tag{9}$$

9. **Shoulder Factor:** The ability of shoulders to shelter disabled vehicles has a strong influence on incident-related delay. Therefore, shoulder widths must be wide enough to store disabled vehicles without them encroaching on the adjacent travel lanes. However, it is possible that narrow shoulders can cause a stopped vehicle to encroach into the adjacent travel lane without causing that lane to be completely blocked. Therefore, values for the Shoulder Factor are computed as a function of shoulder width for right and left shoulders individually:

Shoulder Width	Shoulder Factor (Left and Right)
<= 3 ft	0.0
4-5 ft	0.5
6+ ft	1.0

The shoulder factor for use in the equations is then:

$$\text{ShldFac} = \{\text{SF(left)} + \text{SF(right)}\}/2 \tag{10}$$

10. **Link Length:** The length of the link in miles (to the nearest tenth) should be noted. Link length is used in VMT calculations.

11. **Percent of Annual VMT in the peak period:** This data item is used only if the peak period is used as the time period of analysis. The peak period is defined as weekdays between the hours 6:00 to 10: A.M. and 3:00 to 7:00 P.M. If locally defined values are unavailable, the defaults in Table 3.2 may be used. In developing weekday peak period VMT, it is necessary to account for both weekdays and weekends. A simple approximation would be:

$$\text{PPVMT} = (\text{PPVOL}/\text{AWDT}) * (5/7) * (\text{AWDT}/\text{AADT}) \tag{11}$$

where:

PPVMT = percent of annual VMT in the weekday peak period
PPVOL = average peak period volumes

AWDT	=	annual average weekday traffic
(5/7)	=	ratio of weekdays to total days in the year

As before, an average approximation of AWDT/AADT is 1.0757.

Table 3.2 VMT Proportions for Freeways (Both Directions Combined)

AADT/C	Percent of Traffic in Peak Hour	Percent of Traffic in Peak Period
1	0.0787	0.3844
2	0.0786	0.3844
3	0.0788	0.3847
4	0.0789	0.3852
5	0.0789	0.3845
6	0.0784	0.3842
7	0.0787	0.3844
8	0.0768	0.3830
9	0.0745	0.3814
10	0.0718	0.3777
11	0.0619	0.3720
12	0.0620	0.3644
13	0.0602	0.3497
14	0.0579	0.3339
15	0.0557	0.3188
16	0.0533	0.3045
17	0.0509	0.2925
18	0.0489	0.2823

Step 5: Calculate VMT and Hours per Vehicle-Mile for Uncongested (H_u) and Incident (H_i) Conditions

For each link, H_u and H_i are estimated using the appropriate equations. VMT is calculated as the AADT times the link length in miles if the daily time period is used for the analysis. If the peak period is used, then VMT is AADT times link length times the proportion of VMT during the peak period (use Table 3.2 if local values are unavailable).

Step 6: Calculate the Change in Hi Due to Deviation From the Default Accident Rate (Ha)

If the accident rate of the facility is known, then H_i should be adjusted to account for the actual accident rate of the facility by adding H_a to H_i. Note that in Equation 8 if the actual rate is lower than the default rate, H_a is negative.

Step 7: Calculate Hours per Vehicle (Hr) and Number of Vehicle (Vr) for Each Recurring Bottleneck

For each recurring bottleneck, H_r is calculated using the appropriate equation. V_r is set equal to the AADT of the link if the analysis period is daily. For the peak period, AADT is multiplied by the proportion of VMT during the peak period to derive V_r.

Step 8: Calculate Baseline Vehicle-Hours of Travel (VHT) for Entire Corridor (Extended Segment of Highway)

For the facility being analyzed, the total VHT for the corridor should be computed as:

$$\text{Total VHT} = \sum_{l} ((H_{u(l)} + H_{i(l)}) * VMT_l) + \sum_{b} H_{r(b)} * V_{r(b)} \quad (11)$$

where: *l* refers to individual links and *b* refers to recurring bottlenecks.

If desired, the user can also track the proportion of VHT due to incidents (VHT_i), recurring bottlenecks (VHT_r), and uncongested travel (VHT_u) by breaking out the terms in Equation 11. For any given link:

$$VHTu = Hu * VMT \quad (12)$$
$$VHTi = Hi * VMT \quad (13)$$
$$VHTr = Hr * Vr \quad (14)$$

The analyst is cautioned that VHT_i and VHT_r are measures of systemwide delay due to queuing while VHT_u is the total vehicle-hours of travel for vehicles traversing the segment, and, therefore, is not true delay. The delay incurred by vehicles for unqueued can be found by computing VHT under ideal or "desired" speeds for the segment (e.g., VHT at the free flow speed, VHT_{ffs}) and subtracting it from VHT_u. Then,

$$\text{Total vehicle-hours of delay} = VHT_i + VHT_r + (VHT_u - VHT_{ffs}) \quad (15)$$

Nearly all of the delay imbedded in VHT_u is volume-related: the updated BPR curve predicts noticeable delay when V/C ratios exceed 0.75. Only a small amount of the delay is due to the capacity-reducing effect of incidents. The reason for incident's small influence on unqueued delay is that high volumes occur every day and incidents happen infrequently. The delay portion of VHT_u (the last term in equation 15) can either be kept separate from queued delay or can be counted as recurring delay, ignoring the small (less than one percent) contribution from incidents.

Step 9: Determine the Effects of Incident Management and Transportation Improvement Strategies on the Input Variables

Improvement strategies that have an effect on the input variables in the model will produce different delay estimates. General guidance is provided in Table 3.3 for a variety of improvement types, including capital improvements, incident management, and ITS strategies. Note that for AADT/C, strategies can affect either AADT or capacity. Because it is difficult to quantify specific changes due to strategies in many cases, the user should perform sensitivity analysis using several reasonable levels. For example, in examining the effect of an incident management program, the user might select several different reduction levels for study, say, 10, 20, and 30 percent reductions in incident duration. The equations would then be applied with the original duration factor being reduced by these percents. A brief discussion of the strategies and their affects follows.

Closed Circuit Television (CCTV) Surveillance: Primarily used for incident verification and incident detection to a lesser degree. Main effect will be reduction of incident duration. A possible additional effect is a reduction in secondary accidents due to shorter durations of primary accidents. However, this effect is already partially accounted for within the model and no adjustments are recommended.

Service Patrols: Main effect is the reduction in the incident response component of total incident duration. In some cases, emergency or police vehicles parked on or near the freeway may have a small negative effect on drivers speeds (as reflected by a lower free flow speed) but this effect should not be used unless there is compelling local evidence to the contrary.

Automated Incident Detection: Strategies include incident detection algorithms applied to traffic surveillance data and free cellular phone calls to report incidents. The effect is a reduction in total incident duration.

Computer-Aided Dispatch: This is the automated control of incident response strategies. The effect is a reduction in total incident duration.

Shoulder Widening: The *HCM* reports that shoulder widening up to a point will increase capacity. If the widening is large enough to qualify as a "usable shoulder" for sheltering disabled vehicles, then the shoulder factor will also be affected. Shoulder widening may also lower accident rates, although viable safety relationships for shoulders on urban freeways are scarce.

Interchange Reconfiguration: Reconstructing an interchange can lead to a substantial increase in the capacity of the segment due to elimination of weaving problems and the addition of extra lanes for short distances. The *HCM* should be consulted to estimate the increase in capacity, if any. For many planning applications, detailed before and after capacity calculations based on weaving or complex ramp configurations are not performed the basic freeway lane capacities are used (e.g., Equation 6 above). Therefore, care should be taken in estimating any capacity increase due to interchange configuration as well as many other operational improvements. One approach would be to estimate the percent increase in capacity due to an operational improvement and apply that to the base capacity; even if the absolute value of capacity is off, the relative delay decrease should be representative of actual conditions.

Table 3.3 Effects of Transportation Improvement Strategies on Model Inputs

Strategy	FFS	DurFac	AADT/C	ShldFac	AccRateFac	IncRateFac
CCTV Surveillance	0	**3**	0	0	0	0
Service Patrols	-1[1] - 0	**3**	0	0	0	0
Automated Detection (e.g., free cell phone calls)	0	**3**	0	0	0	0
Computer-aided dispatch for incident response	0	**3**	0	0	0	0
Shoulder widening	0	0	**0-2**	**3**	1	0
Interchange reconfiguration	**0-2**	0	**1-3**	0	**0-3**	0
Accident investigation sites	0	0	0	**1-3**	0	0
Realignment/reconstruction	**0-3**	0	**1-3**	0	**0-2**	0
Safety inspection programs	0	0	0	0	0-1	1-2
Corridor congestion relief	0	0	**1-3**	0	0	0
Demand management	0	0	**1-3**	0	0	0
Lane addition	0	0	**3**	0	0	0
Ramp metering	0	0	**1-3**	0	**0-3**	0
En-route traveler information	0	0	**0-3**	0	1[2]	0

[1] Possible negative effect due to onlooker delay in opposite direction.

+3 to +1 High to low beneficial effect likely
0 No net effect likely
-1 to -3 Low to high adverse effect likely

In some cases, interchange reconfiguration can also lead to a small increase in the free flow speed; if the reconfiguration allows an increase in the posted speed limit, then Equations (1) or (2) can be used to calculate the new free flow speed.

Accident Investigation Sites: In the context of sketch planning, this improvement may be thought of as a shoulder widening to accommodate disabled vehicles involved in an accident (assuming shoulders were not wide enough initially). However, a subset of accidents will be too severe to allow use of these sites. On the other hand, vehicles involved in other incidents (vehicle breakdowns, flat tires) could use the sites as refuge. Examination of crash severity data from the 1993 General Estimates System (GES) show that only 14 percent of urban Interstate crashes involve a fatality, an "incapacitating" injury, or a "nonincapacitating" injury. The remaining 86 percent are either no injury or possible injury; these are the crashes assumed to take advantage of the investigation site. If 25 percent of the remaining (noncrash) incidents is assumed to use the site, the adjusted shoulder factor is:

$$\text{ShldFac}/ = (\text{ShldFac} + N) * \{(0.86 * AR/IR) + (0.25 * (1 - AR/IR))\} \quad (16)$$

where:

ShldFac	=	original shoulder factor
N	=	0.5 if sites are installed on one side of the freeway
	=	1.0 if sites are installed on both sides
AR	=	accident rate (use Table 3.1 as a default)
IR	=	total incident rate, including accident rate (use Table 3.1 as a default)

If local experience indicates that more than 25 percent of noncrash incidents would use the site, then this factor should be increased. For example, consider a situation the original shoulder factor is 0 (no "usable" shoulders), the AADT/C of the link is 10, an accident investigation site is constructed on one side of the freeway, and the default accident and incident rates from Table 3.1 are used. The calculation is:

$$\begin{aligned} \text{ShldFac}/ &= (0 + 0.5) * \{(0.86 * (1.345/9.890)) + (0.25 * (1 - (1.345/9.890)))\} \\ &= 0.5 * \{0.3330) \\ &= 0.1665 \end{aligned}$$

Note that the effect of this improvement is less than if "usable" shoulders were added for the full length of the link, i.e., the adjusted shoulder factor would have been 0.5 in that case.

Realignment/Reconstruction: The same effects as for interchange reconfiguration are indicated.

Vehicle Safety Inspection Programs: These are felt to be the only activity that could possibly have an effect on noncrash incidents. The theory is that by improving vehicle condition, breakdowns are less likely. Unless local evidence suggest otherwise, it is recommended that the effect of these programs be ignored.

Demand Management: These strategies include Transportation Control Measures instituted for air quality reasons as well as many other strategies (e.g., ridesharing). Their effect is to either eliminate trips or to shift them from peak travel times. If trips are eliminated because of demand management, then an estimate should be made of how many of these trips would

be using the facility in question, and AADT should be reduced. If trips are shifted out of the peak hour or peak period to other times, analysis is more difficult. The most direct way of handling this is to reduce the amount of VMT in the peak hour or period, but this is not always possible. Some demand strategies shift trips from the peak **hour** into adjacent hours (peak spreading). However, recall that the peak period for the model is four hours in the morning and four hours in the afternoon. It is therefore not possible to adjust peak period VMT for this case. A similar situation occurs for daily analysis; the shifting of trips will also reduce delay for daily analysis by reducing delays in the peak period. In cases where VMT can not be adjusted, it is recommended that AADT be adjusted as an indirect way of handling the peak VMT shift within the model. For the purpose of planning applications, it can be assumed that the shift affects only recurring delay total daily trips are the same so the same number of incidents can be expected. (This is not strictly true since accident potential in the model is assumed to increase with congestion.) Therefore, AADT in the AADT/C term **for the recurring delay equation only** should be adjusted. For the sake of simplicity, the following recommendations are made:

Effect of Demand Management	Peak Period Analysis	Daily Analysis
Eliminate trips	Reduce AADT by percent of trips eliminated for the facility	Reduce AADT by percent of trips eliminated for the facility
Move trips from peak hour into other hours in the peak period	Reduce AADT in recurring delay eq. only by the percent of trips shifted for the facility	Reduce AADT in recurring delay eq. only by the percent of trips shifted for the facility
Move trips from peak period to off-peak periods	Reduce peak period VMT by the percent of trips shifted for the facility (e.g., Table 3.2)	Reduce AADT in recurring delay eq. only by the percent of trips shifted for the facility

Lane Addition: Basically, a capacity improvement, the same effects as for interchange reconfiguration are indicated.

Ramp Metering: Locations where ramp metering has been installed generally report an increase in capacity and a corresponding decrease in delay **for the freeway**. (Analysts should be aware that additional delay is usually sustained by vehicles on the on-ramp, and spillbacks onto adjacent arterial streets can occur.) There is also some evidence that accident potential is reduced by ramp metering, presumably because of smoother merge operations and less turbulent mainline flow.

En-Route Traveler Information: The effects of traveler information systems are similar to demand management, except that the they tend to be more dynamic. That is, diversions from the freeway only occur when congestion reaches a certain level.

Step 10: Repeat Steps 5 through 8

For the selected strategy(s), repeat the delay calculations and note the difference; this is the delay savings due to the strategy(s).

4.0 Example Application: Knoxville, TN

4.1 Description of Corridor

The corridor selected by the Knoxville MPO for the case study is the most heavily traveled corridor in the region. (The MPO expressed interest in applying the procedure to several more freeway corridors at some time in the future.) The corridor begins at a point three miles west of the CBD at the I-40 and I-75 junction; the two Interstates then run coincidentally throughout the corridor. The corridor ends at the western junction of I-40 and I-75, a total length of 16.80 miles. The study corridor runs through the most rapidly growing area of the Knoxville area. Traffic steadily builds as one progresses from west to east; it essentially doubles from the western-most section to the eastern-most section, indicating heavy use by purely local traffic. Through truck traffic is extremely high: over 25,000 trucks per day have been counted at the weigh station that is located near the western terminus of the corridor. The freeway consists of three basic through lanes in each direction. The corridor is currently undergoing a major reconstruction that consists of adding a through lane in each direction and reconstructing eight of the 10 interchanges. (The I-40/75 junctions at either end of the corridor are not undergoing reconstruction.) A parallel arterial exists in the corridor, State Route 11/70, and has a five-lane cross-section (center two-way left turn lane). A schematic of the corridor appears as Figure 4.1.

4.2 Development of Input Data

The format for this section is the same as shown in Chapter 3.0. The input data is summarized in Table 4.1.

4.2.1 Step 1: Decide if Directional Analysis is Needed

Since the directional distribution of volumes and the number of through lanes are the same in both directions, directional analysis was not undertaken, i.e., the equations for combined directions were used.

Figure 4.1 Schematic Map for the Knoxville Study Corridor

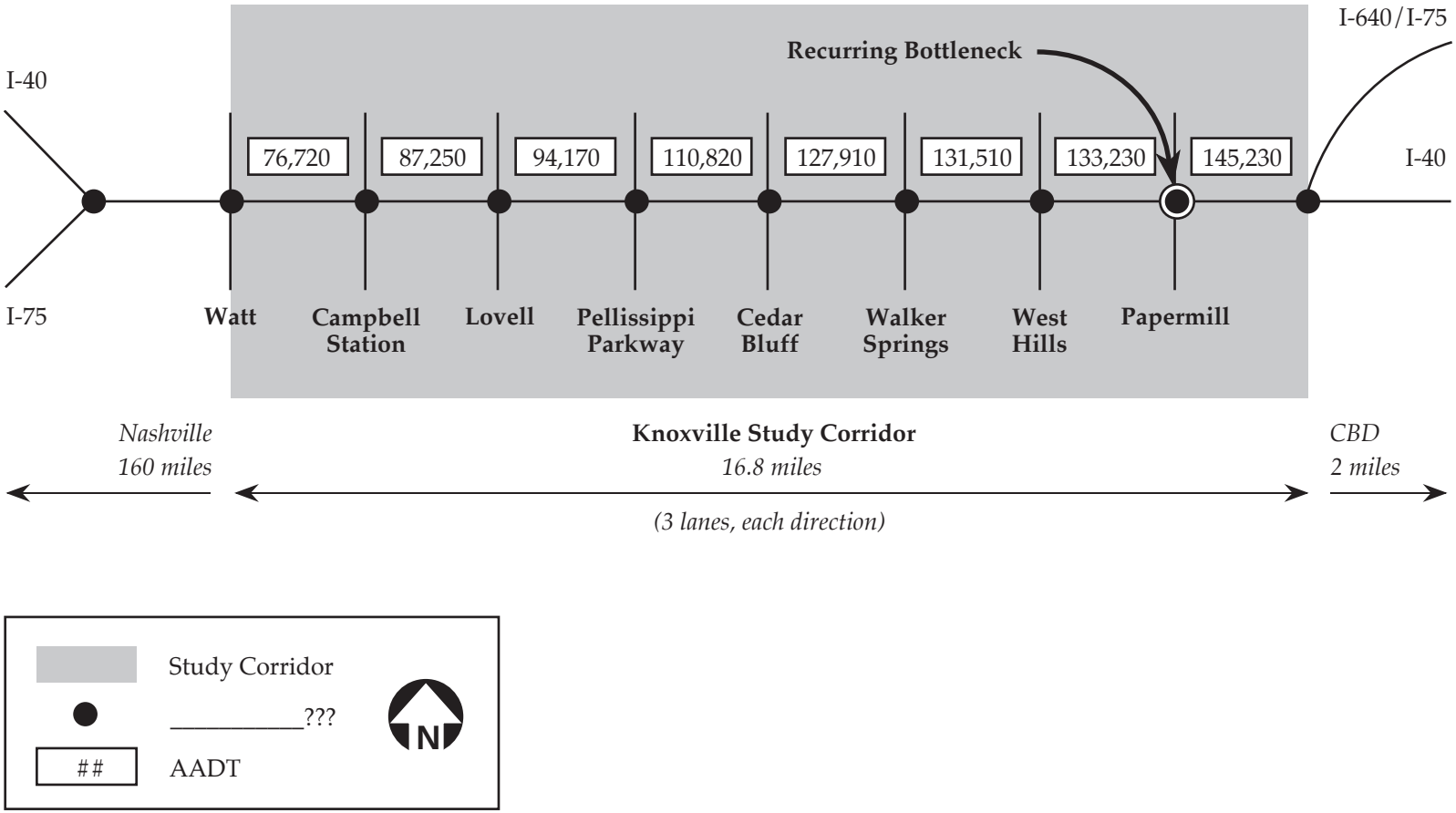


Table 4.1 Basic Data for Knoxville Study Corridor (I-40/75)

Name	AADT	% Peak Period	Speed Limit	# Lanes (one-way)	One-way capacity (vph)	Link Length (miles)	Shoulders	Recurring bottleneck?
I-640 Papermill Dr	145,230	38.14	55	3	6,552	1.90	Both 6'+	Y
Papermill Dr West Hills	133,230	38.14	55	3	6,552	2.80	Both 6'+	N
West Hills Gallaher View	131,510	38.14	55	3	6,552	1.60	Both 6'+	N
Gallaher View Cedar Bluff	127,910	38.14	55	3	6,552	1.20	Both 6'+	N
Cedar Bluff Pellissippi Pky	110,820	38.14	55	3	6,552	2.20	Both 6'+	N
Pellissippi Pky Lovell Rd	94,170	38.14	55	3	6,552	1.70	Both 6'+	N
Lovell Rd Campbell Station Rd	87,250	38.14	55	3	6,552	1.80	Both 6'+	N
Campbell Station Rd Watt Rd	76,270	38.14	70	3	6,552	3.60	Both 6'+	N
						16.80		

Incident Rate, Accident Rate, and Incident Duration set to default values for the base case.

4.2.2 Step 2: Identify Recurring Bottlenecks

The Papermill interchange was identified as the one "true" recurring bottleneck in the corridor. It functions as a bottleneck in both the morning (eastbound) and the afternoon (westbound). The link with the heaviest volume (Papermill to I-640) was selected as the bottleneck link.

4.2.3 Step 3: Define Links

The links were defined by interchange locations because capacity was constant throughout the corridor.

4.2.4 Step 4: Set Input Parameters

For the Base Case, the following input values were used.

1. Analysis Period. Both the peak period and daily timeframes were chosen.

2. Free Flow Speed. The equations predicting free flow speed as a function of speed limit were used.

3. AADT. AADT values were taken directly from Tennessee DOT count data (factored 48-hour counts).

4. Capacity. The capacity equation from Chapter 3.0 was used with a PHF of 0.95.

5. AADT/C. The highest AADT/C ratio was 11.1, well under the critical level of 13.

6. Incident Rate Factor. Default value used.

7. Accident Rate Factor. Default value used.

8. Duration Factor. Default value used.

9. Shoulder Factor. Shoulders on both sides of the traveled way were at least six-feet wide, so the shoulder factor was set at 1.0.

10. Link length. Obtained directly from log files.

11. Percent of Annual VMT in the Peak Period. A value corresponding to an AADT/C of nine was used (38.14 percent) for all links.

4.2.5 Step 5: Calculate VMT and Hours per Vehicle-Mile for Uncongested (H_u) and Incident (H_i) Conditions

Link-based calculations appear in Table 4.2. Note that the equations for three-lane (one direction) freeways were applied.

4.2.6 Step 6: Calculate Change in H_i Due to Deviation From the Default Accident Rate

Since the default accident rate was used, this step is not necessary.

4.2.7 Step 7: Calculate Hours per Vehicle (H_r) and Number of Vehicles (V_r) for Each Recurring Bottleneck

Only the first link (I-640 to Papermill Drive) is designated as a recurring bottleneck; calculations appear in Table 4.2.

4.2.8 Step 8: Calculate Baseline Vehicle-Hours of Travel (VHT) for Entire Corridor

Calculations are made on a link-by-link basis and then summed (Table 4.3).

4.2.9 Step 9: Determine the Effects of Incident Management and Transportation Improvement Strategies

The input data in Table 4.1 were varied to reflect several scenarios as follows.

- Traditional incident management strategies (service patrols coupled with freeway monitoring and incident response protocols); these reduced the default incident duration by 10, 25 and 50 percent.

- Traveler information; AADTs for the segments were reduced by one and 10 percent as an indirect way of accounting for deferred, eliminated, or mode-shifted trips.

- Several strategies based on guidance developed in the Knoxville ITS Early Deployment Plan (EDP). These were:

  - Implementing a basic ITS Infrastructure (incident duration reduced by 41 percent and accident rate reduced by 42 percent {due to elimination of secondary accidents}).

Table 4.2 Calculations for Knoxville Corridor
Part 1

	No Queuing		Queuing		Three Lane Freeway					
	Daily H_u	Pk Per H_u	Daily H_r	Pk Per H_r	Daily H_i	Pk Per H_i	Daily H_i	Pk Per Hi	Daily VMT	Peak Period VMT
I-640 Papermill Dr	0.016408	0.016907	0.020683	0.045975	0.003073	0.004982	0.003073	0.004982	275,937	105,242
Papermill Dr West Hills	0.016272	0.016647	0.000000	0.000000	0.001968	0.003448	0.001968	0.003448	373,044	142,279
West Hills Gallaher View	0.016255	0.016612	0.000000	0.000000	0.001835	0.003250	0.001835	0.003250	210,416	80,253
Gallaher View Cedar Bluff	0.016220	0.016541	0.000000	0.000000	0.001590	0.002881	0.001590	0.002881	153,492	58,542
Cedar Bluff Pellissippi Pky	0.016092	0.016255	0.000000	0.000000	0.000714	0.001436	0.000714	0.001436	243,804	92,987
Pellissippi Pky Lovell Rd	0.016054	0.016033	0.000000	0.000000	0.000297	0.000570	0.000297	0.000570	160,089	61,058
Lovell Rd Campbell Station Rd	0.016039	0.016029	0.000000	0.000000	0.000205	0.000374	0.000205	0.000374	157,050	59,899
Campbell Station Rd Watt Rd	0.013230	0.013228	0.000000	0.000000	0.000109	0.000188	0.000109	0.000188	274,572	104,722
									1,848,404	704,981

Table 4.3 Calculations for Knoxville Corridor
Part 2

	Daily V_r	Peak V_r	Daily VHT_u	Peak Period VHT_u	Daily VHT_i	Peak Period VHT_i	Daily VHT_r	Peak Period VHT_r	Total Daily VHT	Total Peak Period VHT
I-640 Papermill Dr	145,230	55,391	4,528	1,779	848	524	3,004	2,547	8,379	4,850
Papermill Dr West Hills	133,230	50,814	6,070	2,369	734	491	0	0	6,804	2,859
West Hills Gallaher View	131,510	50,158	3,420	1,333	386	261	0	0	3,806	1,594
Gallaher View Cedar Bluff	127,910	48,785	2,490	968	244	169	0	0	2,734	1,137
Cedar Bluff Pellissippi Pky	110,820	42,267	3,923	1,512	174	134	0	0	4,097	1,645
Pellissippi Pky Lovell Rd	94,170	35,916	2,570	979	48	35	0	0	2,618	1,014
Lovell Rd Campbell Station Rd	87,250	33,277	2,519	960	32	22	0	0	2,551	983
Campbell Station Rd Watt Rd	76,270	29,089	3,633	1,385	30	20	0	0	3,663	1,405
	906,390	345,697	29,153	11,285	2,496	1,655	3,004	2,547	34,653	15,487

- A complete incident management system (accident rate reduced by 15 percent, incident duration decreased by 30 minutes, increase AADT by 10 percent to account for induced travel, and increase capacity by 15 percent).
- Transit improvements (reduce incident duration by 15 percent, reduce accident rate by 10 percent).

- Several scenarios to study the effect of workzones on current operations were also run. These alternated single links as the workzones, which replicates the staging of the reconstruction. When a link became a workzone, the capacity was reduced by 10 percent, shoulders were eliminated, and the link was designated as a recurring bottleneck. (The decision to identify links as recurring bottlenecks was based on observations of what is occurring in the corridor now.)

Several of the assumed impacts from the EDP-based strategies appear to be inconsistent or unrealistic. For example, the 30-minute incident duration reduction compared to the default of 38 minutes is a 77 percent decrease! Also, the 15 percent increase in capacity seems like a double counting of benefits given that incident duration has already been reduced. However, the goal was not to measure the exact benefits but rather to test the procedure under a variety of conditions. Therefore, the assumptions do not need to be questioned here. **However, analysts pondering future applications are cautioned to develop changes in the input data judiciously.** In particular, the double counting of expected changes should be avoided.

The base case resulted in the following statistics for the corridor (Table 4.4):

Table 4.4 Base Case Results for Knoxville, I-40/75

	Daily		Peak Period	
	Hours	**Percent**	**Hours**	**Percent**
Delay due to Incidents	2,495.98	7.20	1,654.88	10.69
Delay due to Recurring Bottlenecks	3,003.82	8.67	2,546.58	16.44
Uncongested Vehicle Hours of Travel	29,152.84	84.13	11,285.25	72.87
Total	**34,652.64**	**100.00**	**15,486.71**	**100.00**

Approximately 45 percent of the daily queued delay in the corridor is due to incidents.

As noted in Chapter 3.0, the "uncongested" VHT is the total VHT for vehicles **not** in queues. To calculate the delay portion of this number, the free flow speed and VMT for each link is used to estimate what VHT would be under "ideal" conditions:

"Ideal" VHT = VMT/Free Flow Speed.

As before, the calculations are made for individual links and summed. When this is done, "ideal" VHT is calculated as 28,854 hours for the daily period and 11,004 hours for the peak period. By subtraction, the daily and peak period nonqueued delay values are small: 299 and 280 hours, respectively. These small values are result of using a very flat delay function for V/C levels less than 1.0. (The one used in the model is the modified BPR curve recommended in *NCHRP 387*.)

All of the incident management scenarios reduced the incident-related delay:

- Basic ITS Infrastructure: 65 percent reduction in incident delay.
- 10, 25, and 50 percent reduction in incident duration: 19, 44, and 75 percent reduction in incident delay.
- Complete Incident Management System: 96 percent reduction in incident delay (due to large assumed decrease in duration).
- Transit Improvements: 28 percent reduction in incident delay.
- Traveler Information (decreases in AADT of one and 10 percent): six and 49 percent reduction in incident delay.

More dramatic results (higher delays for both recurring and incident components) were obtained by assigning individual links as workzones one-by-one. The reason for large increases in delay is that the two most sensitive factors of the model were changed: identification of recurring bottlenecks and the shoulder factor. For example, assigning the third link in Table 4.1 as a workzone more than doubled the incident delay *in the entire corridor* and almost doubled recurring delay. Given the base data on lateral location which indicate that around 90 percent of total incidents occur on shoulders these results are not unreasonable. Also, it must be remembered that the entire length of the link was coded as a workzone for simplicity; in reality, the actual workzone will be confined to a portion of the link. For completeness, analysts should divide the original link into at least two new links to reflect the change in conditions (capacity and shoulder width) for the workzone.