AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS

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ERRATA

March 2016

Dear Customer:

Recently, we were made aware of some technical revisions that need to be applied to the *Highway Safety Manual, 1st Edition,* and the *Highway Safety Manual Supplement*.

Changes are reflected by color-coded highlights on individual pages throughout errata file. 2010 changes are highlighted in **blue,** 2012 changes are highlighted in **red,** and 2016 changes are highlighted in **green.**

Please scroll down to see the full erratum.

In the event that you need to download this erratum file again, please download from: http://downloads.transportation.org/HSM-1-M-Errata.pdf.

In order to see color-coding on a hard copy printout, please select "Document and Markups" in the Print dialog box when selecting your print options so that highlighting will be displayed properly.

Then, please replace the existing pages with the corrected pages to ensure that your edition is both accurate and current.

AASHTO staff sincerely apologizes for any inconvenience.

Errata to the *Highway Safety Manual*, 1st Edition and *Supplement*

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Page	Existing Text	Corrected Text
Vol. 1		
2-12	**In Section 2.5, first paragraph, there is a duplicate partial second sentence**	**Paragraph should read:** **"This section considers major road design elements, related driver tasks, and human errors associated with common crash Types. It is not intended to be a comprehensive summary, but is intended to provide examples to help identify opportunities where an understanding of the influence of human factors can be applied to improve design."**
2-17	**In Slow-Moving or Stopped Vehicles Ahead Section change opening of first sentence: "For the controlled-access mainline,…"**	**Sentence should read:** **"As for the controlled-access mainline,…"**
3-17	The term $e^{(-0.4865)}$ is used in Equation 3-4.	Change the term to $e^{(-0.312)}$.
3-21	Answer to Example 1: "Using Equation 3-7, expected crashes . . ."	Change to "Using Equation 3-3 and a calibration factor = 1.0, expected crashes . . ."
3-21	Answer to Example 2: Using Equation 3.8 . . ."	Change the answer to "Using Equation 3.7 . . ."
3-33	2nd to last equation on page began with $\hat{\mu}_{\text{Year 4}} = (5 + 7 - 11 + 9) / (0.87 + 0.71 + 0.64 + 1) =$ $32 / 3.22 = 9.94$ estimate of crashes for the last year:	Change to the following: $\hat{\mu}_{\text{Year 4}} = (5 + 7 + 11 + 9) / (0.87 + 0.71 + 0.64 + 1) =$ $32 / 3.22 = 9.94$ estimate of crashes for the last year:
4-9	**In Table 4-2, third column, Rows 11 and 12: "Expected average crash frequency at the site"**	**Please change text to:** **"Expected average crash frequency per year at the site"**
4-17	The Coefficient of Variation for Segment B1 is shown as $CV_{B1} = \frac{\sqrt{7.7}}{5.7} = 0.53$	The denominator should be 5.2: $CV_{B1} = \frac{\sqrt{7.7}}{5.2} = 0.53$
4-35	In the table showing Ranking Based on RSI, the Average RSI Cost for Intersection 6 is $48,900.	The Average RSI cost for Intersection 6 should be $42,800.
4-42	The calculation of crash frequency variance shown in the table in Step 3 indicates the variance for Signal is 10.5 and the variance for TWSC is 18.8.	The variance for Signal should be 13.75, and the variance for TWSC should be 10.5.
4-46	In Table 4-10, the heading for the last column is "Average 3-Year Expected…"	Change this heading to "Average 3-Year Predicted . . ."

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4-46	Eq. 4-16 is incorrect.	Replace Eq. 4-16 with the following: $\sigma = \sqrt{k N_{predicted}^{2}}$
4-47	**In Table 4-11, all "*N*" should be changed**	**Make the following change to the equations in the table: "$N_{predicted}$"**
4-47	**In Table 4-11, Eq. III:**	**Please make the following change to the equation:** $N_{predicted} \leq N_{observed} < (N_{predicted} + 1.5 \times (\sigma))$
4-55	**In the Alpha and Beta calculations table: $s^2 = 0.034$, α = 0.91, β = 3.2**	**Make the following change in the equation: $s^2 = 0.037$, α = 0.80, β =2.84**
4-57	**In the Ranking Based on Probability table: Wrong values for probability of specific crashes for signalized intersections were provided**	**Corrected values and ranking based on formulas provided**
4-60	In Eq. 4-26, the last item under "where" is $N_{\text{predicted},1(\text{FI})}$.	Change this item to $N_{\text{predicted},n(\text{FI})}$.
7-7	In Section 7.4.3.4, item 1-a-ii is shown as $(P/F,i,y) = (1 + i)(-y)$.	Change this item to $(P/F,i,y) = (1 + i)^{(-y)}$.
7-14	Table 12 is referenced in the last paragraph.	Change the reference to Table 7-5.
7-17	**Equation 7-10:** $\Delta N_{expected(PDO)} = N_{expected(total)} - N_{expected(FI)}$	**Make the following change in the equation:** $\Delta N_{expected(PDO)} = \Delta N_{expected(total)} - \Delta N_{expected(FI)}$
7-17	In Step 4, Eq. 7-13 is shown as: $AM_{(\text{total})} = AM_{(PDO)} \times AM_{(FI)}$.	Change Eq. 7-13 to: $AM_{(\text{total})} = AM_{(PDO)} + AM_{(FI)}$.
7-18	Following Eq. 7-14, at the end of second line of the where list, the $(P/F,i,y)$ is calculated as $(1 + i) - y$.	Change this calculation to $(1 + i)^{(-y)}$.
7-19	In Table 7-10, Columns 2 and 4 need to be revised, as does the heading for Column 2.	Substitute Table 7-10 with the attached revised table.
7-19	The first line under Results states, "The estimated present value monetary benefit of installing a roundabout at Intersection 2 is $33,437,850."	Change the amount to $5,675,500.

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9-8	Step 2 of Figure 9-2 states, "Calculate the predicted crash frequency for each site summed over the entire before period."	Change "predicted" to "expected."
9-21	Eq. A-10 is referenced in the last column in the table under Step 6.	Change the reference to Eq. 9A.1-11.
9-36	**In Equation 9A.1-5:** **"OR_i = Odd Ration at site *i*"**	**Make the following text change:** **"OR_i = Odd Ratio at site *i*"**
9-41	Eq. 9A.2-16 is shown as Safety Effectiveness = 100 × (1 – *R*).	Change this equation to Safety Effectiveness = 100 × (1 – *OR*).

Vol. 2

Page	Existing Text	Corrected Text
C-14	After Figure C-3, a heading was inadvertently omitted.	Add the heading "C.6.3 Safety Performance Functions (SPFs)" immediately following Figure C-3.
C-14	The term $e^{(-0.4865)}$ is used in Eq. C-4.	Change the term to $e^{(-0.312)}$.
C-15	C.6.3 Crash Modification Factors (CMFs)	Renumber the heading as "C.6.4 Crash Modification Factors (CMFs)."
C-18	C.6.4 Calibration of Safety Performance Functions to Local Conditions	Renumber the heading as "C.6.5 Calibration of Safety Performance Functions to Local Conditions."
C-18	C.6.5 Weighting Using the Empirical Bayes Method	Renumber the heading as "C.6.6 Weighting Using the Empirical Bayes Method."
10-17	**In Table 10-3 Note:** **"Accident…Exhibit 10-4."**	**Remove the note**
10-23	The 3rd row about (CMF_{3r}) in Table 10-7 references Table 10-7.	Change this reference to Equation 10-13.
10-23	The 6th row about (CMF_{6r}) in Table 10-7 references Table 10-11.	Change this reference to Equation 10-17.
10-25	The calculation 0.98 + 6.875 is used in an equation within the last row in Table 10-9.	Change the calculation to 0.98 – 6.875.

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10-26	**In the last paragraph, first sentence: "…Table 9, Figure 8, and Table 10…"**	**Make the following text change: "…Tables 10-9 and 10-10, and Figure 10-8…"**
10-42	A 1,2000-ft horizontal curve radius is used in the 5th bullet under the heading "The Facts".	Change the horizontal curve radius to 1,200 ft.
11-10	The second sentence of Step 12 reads, "Otherwise, proceed to Step 14."	Change "Step 14" to "Step 13."
11-11	**In the first definition: "…rural two-lane, two-way road facility…"**	**Make the following text change: "…rural multi-lane highway…"**
11-16	**In the first paragraph, first sentence: "…in Table 11-3…"**	**Make the following change: "…in Table 11-4…"**
11-18	**In the second paragraph, second sentence: "The SPFs for undivided roadway…"**	**Make the following text change: "The SPFs for divided roadway…"**
11-19	**In the first paragraph, first sentence: "The default proportions in Table 11-5…"**	**Make the following Table number change: "The default proportions in Table 11-6…"**
11-31	Paragraph 1, line 7 refers to Table 13-9.	Change this reference to Table 13-13.
11-33	Eq. 11-18 is shown as $CMF_{1i} = \frac{0.016 \times skew}{(0.98 + 0.16 \times skew)} + 1.0$	Change the denominator as follows: $CMF_{1i} = \frac{0.016 \times skew}{(0.98 + 0.016 \times skew)} + 1.0$
11-33	Eq. 11-19 is shown a $CMF_{1i} = \frac{0.017 \times skew}{(0.52 + 0.17 \times skew)} + 1.0$	Change the denominator as follows: $CMF_{1i} = \frac{0.017 \times skew}{(0.52 + 0.017 \times skew)} + 1.0$
11-34	Eq. 11-20 is shown as $CMF_{1i} = \frac{0.053 \times skew}{(1.43 + 0.53 \times skew)} + 1.0$	Change the denominator as follows: $CMF_{1i} = \frac{0.053 \times skew}{(1.43 + 0.053 \times skew)} + 1.0$

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11-34	Eq. 11-21 is shown as $$CMF_{1i} = \frac{0.048 \times skew}{(0.72 + 0.48 \times skew)} + 1.0$$	Change the denominator as follows: $$CMF_{1i} = \frac{0.048 \times skew}{(0.72 + 0.048 \times skew)} + 1.0$$
11-35	**In the CMF_{4i}—Lighting section: "This CMF applies to total intersections crashes (not including vehicle-pedestrian and vehicle-bicycle collisions).**	**Make the following text change: "This CMF applies to total intersection crashes."**
11-50	Step 10, Intersection Skew Angle calculates CMF_{1i} as follows: $$CMF_{1i} = \frac{0.016 \times skew}{(0.98 + 0.16 \times skew)} + 1.0$$ $$CMF_{1i} = \frac{0.016 \times 30}{(0.98 + 0.16 \times 30)} + 1.0 = 1.08$$	Change this calculation to: $$CMF_{1i} = \frac{0.016 \times skew}{(0.98 + 0.016 \times skew)} + 1.0$$ $$CMF_{1i} = \frac{0.016 \times 30}{(0.98 + 0.016 \times 30)} + 1.0 = 1.08$$
11-50	The final calculation in Step 10 is: $CMF_{comb} = 1.08 \times 0.56 \times 0.90 = 0.54$.	Change this calculation to $CMF_{comb} = 1.33 \times 0.56 \times 0.90 = 0.67$.
11-50	Step 11, Calculation of Predicted Average Crash Frequency, indicates the results are = 0.928 × 1.50 × (0.54) = 0.752 crashes/year.	Change these results to = 0.928 × 1.50 × (0.67) = 0.933 crashes/year.
11-52	Column 2 of the table for Worksheet SP3B lists CMF for Intersection Skew Angle for the Total Crash Severity Level in Row 1 as 1.08 and the Fatal and Injury Crash Severity Level in Row 2 as 1.09.	Change the CMF for Intersection Skew Angle for the Total Crash Severity Level to 1.33 and the Fatal and Injury Crash Severity Level to 1.50.
11-52	Column 6 of the table for Worksheet SP3B lists Combined CMF for the Total Crash Severity Level in Row 1 as 0.54 and the Fatal and Injury Crash Severity Level in Row 2 as 0.44.	Change the Combined CMF in Column 6 to 0.67 for the Total Crash Severity Level in Row 1 and to 0.61 for the Fatal and Injury Crash Severity Level in Row 2.
11-52	Column 5 of the Worksheet SP3C show the values for Combined CMFs are 0.54 for Total Crash Severity Level in Row 1, 0.44 for Fatal and Injury Crash Severity Level in Row 2, and 0.44 for Fatal and Injury Crash Severity Level in Row 3.	Change these values to 0.67, 0.61, and 0.61, respectively.

Page	Existing Text	Corrected Text
11-52	Column 7 of the Worksheet SP3C show the values for Predicted Crash Frequency are 0.752 for Total Crash Severity Level in Row 1, 0.286 for Fatal and Injury Crash Severity Level in Row 2, 0.178 for Fatal and Injury Crash Severity Level in Row 3, and 0.466 for Property Damage Only in Row 4.	Change these values to 0.933, 0.396, 0.247, and 0.537, respectively.
11-53	The values in Columns 3, 5, 7, and 9 in the table for Worksheet SP3D are incorrect.	Substitute the table for Worksheet SP3D with the attached revised table.
11-53	In the table for Worksheet SP3E, the values for Predicted Average Crash Frequency listed in Column 2 are 0.752, 0.286, 0.178, 0.466 crashes/year.	Change these values to 0.933, 0.396, 0.247, and 0.537, respectively.
11-55	The values in Columns 2, 3, 4, 7, and 8 for Intersections shown in Worksheet SP4A are incorrect.	Substitute the table for Worksheet SP4A with the attached revised table.
11-55	The final calculation for Intersection 1 is shown as: $w = \frac{1}{1+0.460\times(0.752)} = 0.743$	Change this calculation to $w = \frac{1}{1+0.460\times(0.933)} = 0.700$
11-56	Under the subheading Column 8—Expected Average Crash Frequency, the calculation for Intersection 1 is shown as follows: $N_{expected} = 0.743 \times 0.752 + (1 - 0.743) \times 3 = 1.330$.	Change this calculation to $N_{expected} = 0.700 \times 0.933 + (1 - 0.700) \times 3 = 1.554$.
11-56	The values and the reference to Worksheet 3A in the table for Worksheet SP4B are incorrect.	Substitute the table for Worksheet SP4B with the attached revised table.
11-58	The headings and values shown in the tables for Worksheet SP5A and SP5A Continued are incorrect.	Substitute the tables for Worksheet SP5A and SP5A Continued with the attached revised table.
11-58	**In Worksheet SP5A: there are subhead errors in columns 1-7**	**Revert back to original edition's subhead values**

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11-59	The calculation for w_0 in Column 9 is shown as $= \frac{1}{1+\frac{1.968}{4.347}}$ $= 0.688$	Change the calculation to $= \frac{1}{1+\frac{2.109}{4.528}}$ $= 0.682$
11-59	The calculation for N_0 in Column 10 is shown as $= 0.688 \times 4.347 + (1 - 0.688) \times 9 = 5.799$.	Change the calculation to $0.682 \times 4.528 + (1 - 0.682) \times 9 = 5.950$.
11-59	The calculation for w_1 in Column 11 is shown as $= \frac{1}{1+\frac{2.009}{4.347}}$ $= 0.684$	Change this calculation to $= \frac{1}{1+\frac{2.076}{4.528}}$ $= 0.686$
11-59	The calculation for N_1 in Column 12 is shown as $= 0.684 \times 4.347 + (1 - 0.684) \times 9 = 5.817$.	Change this calculation to $= 0.686 \times 4.528 + (1 - 0.686) \times 9 = 5.932$.
11-59	The calculation for $N_{expected/comb}$ in Column 13 is shown as: $= \frac{5.799+5.817}{2}$ $= 5.808$	Change this calculation to: $= \frac{5.950+5.932}{2}$ $= 5.941$
11-60	The values shown in the table for Worksheet SP5B are incorrect.	Substitute the table for Worksheet SP5B with the attached revised table.
11-61	The first sentence in the section on Results reads, "The predicted average crash frequency for the proposed four-lane facility project is 4.4 crashes per year, and the predicted crash reduction for the project is 8.1 crashes per year."	Change the numbers to 4.5 crashes per year for the predicted average crash frequency and 7.8 crashes per year for the predicted crash reduction.
11-61	The values in Table 11-26 are incorrect.	Substitute Table 11-26 with the attached revised table.
12-10	**In the 12th bullet:** • **"For all intersections…"**	**Remove the bullet**

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12-10	**In the 16th bullet: "Number of major-road approaches with left-turn signal phasing (0, 1, or 2) (signalized intersections only) and type of left-turn signal phasing (permissive, protected/permissive, permissive/protected, or protected)."**	**Make the following text change: "Number of approaches with left-turn signal phasing (0, 1, 2, 3, or 4) (signalized intersections only) and type of left-turn signal phasing (permissive, protected/permissive, permissive/protected, or protected)."**
12-10	**At the end of the page, there is a missing bullet**	**Add bullet text: • Pedestrian volumes**
12-12	The second sentence of Step 12 reads, "Otherwise, proceed to Step 14."	Change "Step 14" to "Step 13."
12-15	**In the middle of the page there is a missing bullet**	**Add bullet text after "Presence/type of median": • Presence of TWLTL**
12-15	**At the end of the page, there is a missing bullet**	**Add bullet text after "Speed" category: • Automated Enforcement**
12-28	**In the last paragraph, first sentence: "The SPF for each of the four intersection…"**	**Make the following change in the text by adding this to the end of the sentence: "…and intersection-related crashes."**
12-41	**In the last paragraph, first sentence: "The CMF applies…cross-median collusions…"**	**Make the following text change to: "The CMF applies…cross-median collisions…"**
12-43	Paragraph 1, line 2 of Section 12.7.2 refers to CMF_{4i}.	Change CMF_{4i} to CMF_{6i}.
12-86	Paragraph 1, Line 1 of the section on Results refers to "the unsignalized intersection in Sample Problem 4."	Change "unsignalized" to "signalized."
12-114	Column head (7) of Worksheet 2C Continued refers to Worksheet SP4B.	Change the reference to Worksheet 2B.
12-115	Column head (7) of Worksheet 2E Continued refers to Worksheet SP4B.	Change the reference to Worksheet 2B.
A-5–A-7	**In Table A-2: "Average Annual Daily Traffic"**	**Make this change throughout the table: "Annual Average Daily Traffic"**

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A-5	In Table A-2, the row for Presence of Lighting for Chapter 11 shows the "X" in the Required column	The "X" should be in the Desirable column.
A-5	In Table A-2, the row for Sideslope for Chapter 11 shows the "X" not aligned in the Required column.	The "X" should be in the Required column.
A-5	**In Table A-2 Rural Multilane Highways, "For divided highways only:"**	**Add italics: "*For divided highways only:*"**
A-6	**In Table A-2, Urban and Suburban Arterials: "Low-speed vs. intermediate or high speed"**	**Please change to the following: "Posted speed limit"**
A-6	In Table A-2, the row for Presence of Lighting for Chapter 11 reads "Need actual datad."	The final d should be deleted so that the text reads "Need actual data."
A-6	In Table A-2, the row for Intersection Skew Angle for Chapter 11 does not have a superscript.	Insert a superscript "d" to indicate a cross reference to table note d. The text should read, "Assume no skew^{d}"
A-7	In Table A-2, Pedestrian Volume for Chapter 12 refers to "Estimate with Table 12-21."	Change the cross reference to Table 12-15.
A-8	The first line of Example Calibration Factor Calculation refers to Equation 10-18.	Change the cross reference to Equation 10-10.
A-8	**In Equation 10-10:** $N_{spf}\ \text{int} = e^{[-5.73...]}$	**Make the following change in the equation:** $N_{spf}\ \text{int} = e^{[-5.13...]}$
A-8	In the calculation for predicted crash frequency, the sample indicates that $N_{bibase} = e^{(-5.73+60\times\ln(4000)+0.20\times\ln(2000)} = 3.922$ crashes/year	The correct values should be $N_{bibase} = e^{(-5.13+60\times\ln(4000)+0.20\times\ln(2000)} = 3.922$ crashes/year
A-8	In the paragraph following the N_{bibase} calculation, the first sentence states, "The intersection has a left-turn lane on the major road, for which CMF_{1i} is 0.67, and a right-turn lane on one approach, a feature for which CMF_{2i} is 0.98."	Change this text to read: "The intersection has a left-turn lane on the major road, for which CMF_{1i} is 0.82, and a right-turn lane on one approach, a feature for which CMF_{2i} is 0.96."

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A-8	In the last equation on the page, the calculation for predicted average crash frequency is recorded as "2.152 × 0.67 × 0.98 × 3 = 4.240 crashes in three years, shown in Column 9."	The correct values for this calculation are actually "3.922 × 0.82 × 0.96 × 3 = 9.262 crashes in three years, shown in Column 9."
A-8	The final paragraph, second sentence on page A-8 indicates, ". . . The sum of the observed crash frequencies in Column 10 (43) is divided by the sum of the predicted average crash frequencies in Column 9 (45.594) to obtain the calibration factor, C_i, equal to 0.943. It is recommended that calibration factors be rounded to two decimal places, so calibration factor equal to 0.94 should be used . . ."	These sentences of the paragraph should read, ". . .The sum of the observed crash frequencies in Column 10 (43) is divided by the sum of the predicted average crash frequencies in Column 9 (87.928) to obtain the calibration factor, C_i, equal to 0.489. It is recommended that calibration factors be rounded to two decimal places, so calibration factor equal to 0.49 should be used . . ."
A-9	The values in Columns 3, 5, 7, and 9 of Table Ex-1 are incorrect.	Substitute the table for Table Ex-1 with the attached revised table.
A-11	**In Table A-3, Table 11-19 Intersections "X"**	**Make the following change in the table: Table 11-19 Roadway Segments "X"**
A-14	**In the paragraph, Crash Severity and Collision Type for Single-Vehicle Crashes by Intersection Type**	**Add the following text: The default values for f_{bisv} in Equation 12-27 should be replaced with locally available data.**
A-14	**In the last paragraph, second sentence: "For a given facility type and speed category,…"**	**Make the following text change: "For a given facility type,…"**
A-20	**In Eq. A-9:** $$N_{predicted\,w1} = \sum_{j=1}^{5}\sqrt{k_{rmj}\,N_{rmj}} + \sum_{j=1}^{5}\sqrt{k_{rsj}\,N_{isj}} + \sum_{j=1}^{5}\sqrt{k_{rdj}\,N_{rdj}} + \sum_{j=1}^{4}\sqrt{k_{imj}\,N_{imj}} + \sum_{j=1}^{4}\sqrt{k_{isj}\,N_{isj}}$$	**Make the following change in the equation:** $$N_{predicted,\ w1} = \left[\sum_{j=1}^{5}\sqrt{k_{rmj}\,N_{rmj}^2} + \sum_{j=1}^{5}\sqrt{k_{rsj}\,N_{rsj}^2} + \sum_{j=1}^{5}\sqrt{k_{rdj}\,N_{rdj}^2} + \sum_{j=1}^{4}\sqrt{k_{imj}\,N_{imj}^2} + \sum_{j=1}^{4}\sqrt{k_{isj}\,N_{isj}^2}\right]^2$$

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Vol. 3		
D-3	A sentence for Row "T" in Table D-1 was omitted.	Add "A list of these treatments is presented in the appendices to each chapter" as the final sentence in this row.
D-5	Item 2 in Section D.4.4, Application of CMFs to Estimate Crash Frequency, refers to "…an expected number of crashes…"	Change this phrase to "…a predicted number of crashes…"
13-22	There is no specific table referenced in the first sentence of the first paragraph to Section 13.5.2.2.	Reference Table 13-21.
13-24	Table 13-2 is referenced in the 2nd paragraph to Section 13.5.2.4.	Change the reference to Table 13-23.
13-35	Table 13-411 is referenced in the 2nd paragraph to Section 13.8.2.7.	Change the reference to Table 13-41.
13-48	Table 13-54 is missing a row for installing pedestrian-activated flashing yellow beacons with overhead signs. In addition, the treatment to provide pedestrian overpasses and underpasses has a trend for urban arterials.	Replace Table 13-54 with the attached revision.
13-51	**Figure 13-11 was calculated using the metric coefficient 0.2**	**Make the following change to the figure: Figure calculated using the converted coefficient 0.322**
14-10	Exhibit 14-8 is referenced in the last paragraph.	Change the reference to Table 14-4.
14-19	Eq. 14-3 is shown as $CMF_{1i} = \frac{0.016 \times skew}{(0.98 + 0.16 \times skew)} + 1.0$	Change the denominator as follows: $CMF_{1i} = \frac{0.016 \times skew}{(0.98 + 0.016 \times skew)} + 1.0$
14-19	Eq. 14-4 is shown as $CMF_{1i} = \frac{0.053 \times skew}{(1.43 + 0.53 \times skew)} + 1.0$	Change the denominator as follows: $CMF_{1i} = \frac{0.053 \times skew}{(1.43 + 0.053 \times skew)} + 1.0$
14-20	Eq. 14-5 is shown as $CMF_{1i} = \frac{0.017 \times skew}{(0.52 + 0.17 \times skew)} + 1.0$	Change the denominator as follows: $CMF_{1i} = \frac{0.017 \times skew}{(0.52 + 0.017 \times skew)} + 1.0$

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14-20	Eq. 14-6 is shown as $CMF_{1i} = \frac{0.048 \times skew}{(0.72 + 0.48 \times skew)} + 1.0$	Change the denominator as follows: $CMF_{1i} = \frac{0.048 \times skew}{(0.72 + 0.048 \times skew)} + 1.0$
14-42	Exhibit 14-38 is referenced in the 3rd paragraph to Section 14.7.2.8.	Change the reference to Table 14-28.
14-52	The final sentence of Section 14A.5.1.8 reads, "The LPI provides pedestrians an opportunity to begin crossing without concern for turning vehicles (assuming right-on-red is <u>permitted</u>)."	This sentence should read, "The LPI provides pedestrians an opportunity to begin crossing without concern for turning vehicles (assuming right-on-red is <u>prohibited</u>)."
15-4	There are unknown crash effects for modifying ramp type or configuration and for providing pedestrian facilities on ramp terminals.	Replace Table 15-1 with the attached revision. The two rows referring to modifying ramp type or configuration and to providing pedestrian facilities on ramp terminals have been deleted.
15-6	**Eq. 15-3:** $CMF = \frac{1.576 \times e^{(-4.55 \times 0.12)}}{1.576 \times e^{(-4.55 \times 0.20)}} = 0.69$	**Revise the equation:** $CMF = \frac{1.576 \times e^{(-4.55 \times 0.20)}}{1.576 \times e^{(-4.55 \times 0.12)}} = 0.69$
15-11	A bulleted item is missing under Section 15A.3.1, Ramp Roadways.	Add the following bulleted item: ■ Modify ramp type or configuration.
15-12	A bulleted item is missing under Section 15A.3.1, Bicyclists and Pedestrians.	Add the following bulleted item: ■ Provide pedestrian facilities on ramp terminals.
16-6	Exhibit 16-5 is referenced in the 2nd sentence to Section 16.4.2.1.	Change the reference to Figure 16-1.
16-6	Figure 16-1 is referenced in the 3rd sentence to Section 16.4.2.1.	Change the reference to Figure 16-2.

Page	Existing Text	Corrected Text
17-5	There are unknown crash effects for mitigating aggressive driving through engineering and for implementing older driver education and retesting programs.	Replace Table 17-4 with the attached revision. The two rows referring to mitigating aggressive driving through engineering and to implementing older driver education and retesting programs have been deleted.
17-14	A subsection is missing under Section 17A.4.1.	Add the following subsection and text after the 3rd paragraph to subsection 17A.4.1.6: 17A.4.1.7. Conduct Enforcement to Reduce Red-Light Running Automated enforcement for red-light running, combined with appropriate enabling legislation, potentially reduces crashes.
17-14 to 17-16	As a result of adding a subsection that was missing under Section 17A.4.1, renumber the remaining subsections.	Renumber as follows: ▪ Section 17A.4.1.7 as Section 17A.4.1.8 ▪ Section 17A.4.1.8 as Section 17A.4.1.9 ▪ Section 17A.4.1.9 as Section 17A.4.1.10 ▪ Section 17A.4.1.10 as Section 17A.4.1.11 ▪ Section 17A.4.1.11 as Section 17A.4.1.12

Supplement

18-42	**In Eq. 18-35:** $CMF_{8,\ fs,ac,sv,z} = \left(1.0 - \sum_{i=1}^{m} P_{c,i} \times f_{c,i}\right) \times \exp\left(a \times [W_s - 10]\right) + \left(\sum_{i=1}^{m} P_{c,i} \times f_{c,i}\right) \times \exp\left(b \times [W_s - 10]\right)$	**Make the following change to the equation:** $CMF_{8,\ fs,ac,sv,z} = \left(1.0 - \sum_{i=1}^{m} P_{c,i}\right) \times \exp\left(a \times [W_s - 10]\right) + \left(\sum_{i=1}^{m} P_{c,i}\right) \times \exp\left(b \times [W_s - 10]\right)$

Errata to the *Highway Safety Manual*, 1st Edition and *Supplement*

• 2010 errata changes in blue • 2012 errata changes in red • 2016 errata changes in **green (bold)**

Page	Existing Text	Corrected Text
18-43	**In Eq. 18-36:** $$CMF_{9,\ fs,ac,sv,\ fi} = \left(1.0 - \sum_{i=1}^{m} P_{c,i} \times f_{c,i}\right) \times f_{\tan} + \left(\sum_{i=1}^{m} P_{c,i} \times f_{c,i}\right) \times 1.0$$	**Make the following change to the equation:** $$CMF_{9,\ fs,ac,sv,\ fi} = \left(1.0 - \sum_{i=1}^{m} P_{c,i}\right) \times f_{\tan} + \left(\sum_{i=1}^{m} P_{c,i}\right) \times 1.0$$
18-52	**In the first paragraph, last sentence; and the third paragraph, last sentence: "…is less than 0.75 ft, then it should…"**	**Make the following text change: "…is less than 0.75 ft, then W_{icb} should…"**
19-67	**Following Table 19-42**	**Please add the following text: "The curve entry speeds need to be calculated for all curves from milepost 0.0 to the end of the analysis segment. This may include segments of an adjacent ramp that are not included in the current analysis segment. For each curve, record the entry speed, the total length of the curve, and the length of the current analysis segment. Once the procedure on the following pages is completed, return to Equation 19-33. In this equation, the summation term only includes entry speeds and radii that have a length in the current analysis segment. All other curves analyzed should be ignored if they are not part of the current analysis segment."**
Throughout	Some equations using natural logarithm were inadvertently typeset with "In" instead of "ln".	Please replace the "In" with "ln" in these equations.

information in a timely fashion, when they are overloaded with information, or when their expectations are not met, slowed responses and errors may occur.

Design that conforms to long-term expectancies reduces the chance of driver error. For example, drivers expect that there are no traffic signals on freeways and that freeway exits are on the right. If design conforms to those expectancies it reduces the risk of a crash. Short-term expectancies can also be impacted by design decisions. An example of a short-term expectation is that subsequent curves on a section of road are gradual, given that all previous curves were gradual.

With respect to traffic control devices, the positive guidance approach emphasizes assisting the driver with processing information accurately and quickly by considering the following:

- *Primacy*—Determine the placements of signs according to the importance of information, and avoid presenting the driver with information when and where the information is not essential.
- *Spreading*—Where all the information required by the driver cannot be placed on one sign or on a number of signs at one location, spread the signage along the road so that information is given in small chunks to reduce information load.
- *Coding*—Where possible, organize pieces of information into larger units. Color and shape coding of traffic signs accomplishes this organization by representing specific information about the message based on the color of the sign background and the shape of the sign panel (e.g., warning signs are yellow, regulatory signs are white).
- *Redundancy*—Say the same thing in more than one way. For example, the stop sign in North America has a unique shape and message, both of which convey the message to stop. A second example of redundancy is to give the same information by using two devices (e.g., "no passing" indicated with both signs and pavement markings).

2.5. IMPACTS OF ROAD DESIGN ON THE DRIVER

This section considers major road design elements, related driver tasks, and human errors associated with common crash types. It is not intended to be a comprehensive summary, but is intended to provide examples to help identify opportunities where an understanding of the influence of human factors can be applied to improve design.

2.5.1. Intersections and Access Points

As discussed in Section 2.2, the driving task involves control, guidance, and navigation elements. At intersections, each of these elements presents challenges:

- *Control*—The path through the intersection is typically unmarked and may involve turning;
- *Guidance*—There are numerous potential conflicts with other vehicles, pedestrians, and cyclists on conflicting paths; and
- *Navigation*—Changes in direction are usually made at intersections, and road name signing can be difficult to locate and read in time to accomplish any required lane changes.

In the process of negotiating any intersection, drivers are required to:

- Detect the intersection;
- Identify signalization and appropriate paths;
- Search for vehicles, pedestrians, and bicyclists on a conflicting path;
- Assess adequacy of gaps for turning movements;
- Rapidly make a stop/go decision on the approach to a signalized intersection when in the decision zone; and
- Successfully complete through or turning maneuvers.

Slow-Moving or Stopped Vehicles Ahead

As for the controlled-access mainline, rear-end and sideswipe crashes occur when drivers encounter unexpected slowing or stopped vehicles and realize too late their closing speed.

Poor Visibility of Vulnerable Road Users or Animals

Vulnerable road user and animal crashes may occur due to low contrast with the background and drivers' inability to detect pedestrians, cyclists, or animals in time to stop.

2.6. SUMMARY—HUMAN FACTORS AND THE HSM

This chapter described the key factors of human behavior and ability that influence how drivers interact with the roadway. The core elements of the driving task were outlined and related to human ability so as to identify areas where humans may not always successfully complete the tasks. There is potential to reduce driver error and associated crashes by accounting for the following driver characteristics and limitations described in the chapter:

- *Attention and information processing*—Drivers can only process a limited amount of information and often rely on past experience to manage the amount of new information they must process while driving. Drivers can process information best when it is presented in accordance with expectations, sequentially to maintain a consistent level of demand, and in a way that it helps drivers prioritize the most essential information.
- *Vision*—Approximately 90 percent of the information used by a driver is obtained visually (17). It is important that the information be presented in a way that considers the variability of driver visual capability so that users can see, comprehend, and respond to it appropriately.
- *Perception-reaction time*—The amount of time and distance needed by one driver to respond to a stimulus (e.g., hazard in road, traffic control device, or guide sign) depends on human elements, including information processing, driver alertness, driver expectations, and vision.
- *Speed choice*—Drivers use perceptual and road message cues to determine a speed they perceive to be safe. Information taken in through peripheral vision may lead drivers to speed up or slow down depending on the distance from the vehicle to the roadside objects (38). Drivers may also drive faster than they realize after adapting to highway speeds and subsequently entering a lower-level facility (37).

Knowledge of both engineering principles and the effects of human factors can be applied through the positive guidance approach to road design. The positive guidance approach is based on the central principle that road design that corresponds with driver limitations and expectations increases the likelihood of drivers responding to situations and information correctly and quickly. When drivers are not provided or do not accept information in a timely fashion, when they are overloaded with information, or when their expectations are not met, slowed responses and errors may occur.

An understanding of human factors and their affects can be applied to all projects regardless of the project focus. Parts B, C, and D provide specific guidance on the roadway safety management process, estimating safety effects of design alternatives, and predicting safety on different facilities. Considering the effect of human factors on these activities can improve decision making and design considerations in analyzing and developing safer roads.

2.7. REFERENCES

(1) Alexander, G. J. and H. Lunenfeld. *Driver Expectancy in Highway Design and Traffic Operations*. Publication No. FHWA-TO-86-1. Federal Highway Administration, U.S. Department of Transportation, Washington, DC. 1986.

(2) Alexander, G. and H. Lunenfeld. Positive guidance in traffic control. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1975.

While the functional form of the SPFs varies in the HSM, the predictive model to estimate the expected average crash frequency Npredicted, is generally calculated using Equation 3-3.

$$N_{predicted} = N_{SPF\,x} \times (CMF_{1x} \times CMF_{2x} \times \ldots CMF_{yx}) \times C_x \quad (3\text{-}3)$$

Where:

$N_{predicted}$ = predictive model estimate of crash frequency for a specific year on site type *x* (crashes/year);

$N_{SPF\,x}$ = predicted average crash frequency determined for base conditions with the Safety Performance Function representing site type *x* (crashes/year);

CMF_{yx} = Crash Modification Factors specific to site type *x*;

C_x = Calibration Factor to adjust for local conditions for site type *x*.

The HSM provides a detailed predictive method for the following three facility types:

- *Chapter 10*—Rural Two-Lane Two-Way Roads;
- *Chapter 11*—Rural Multilane Highways;
- *Chapter 12*—Urban and Suburban Arterials.

Advantages of the Predictive Method

Advantages of the predictive method are that:

- Regression-to-the-mean bias is addressed as the method concentrates on long-term expected average crash frequency rather than short-term observed crash frequency.
- Reliance on availability of limited crash data for any one site is reduced by incorporating predictive relationships based on data from many similar sites.
- The method accounts for the fundamentally nonlinear relationship between crash frequency and traffic volume.
- The SPFs in the HSM are based on the negative binomial distribution, which are better suited to modeling the high natural variability of crash data than traditional modeling techniques based on the normal distribution.

First-time users of the HSM who wish to apply the predictive method are advised to read Section 3.5 (this section), read the Part C—Introduction and Applications Guidance, and then select an appropriate facility type from Chapters 10, 11, or 12 for the roadway network, facility, or site under consideration.

3.5.2. Safety Performance Functions

Safety Performance Functions (SPFs) are regression equations that estimate the average crash frequency for a specific site type (with specified base conditions) as a function of annual average daily traffic (AADT) and, in the case of roadway segments, the segment length (L). Base conditions are specified for each SPF and may include conditions such as lane width, presence or absence of lighting, presence of turn lanes, etc. An example of an SPF (for roadway segments on rural two-lane highways) is shown in Equation 3-4.

$$N_{SPF\,rs} = (AADT) \times (L) \times (365) \times 10^{(-6)} \times e^{(-0.312)} \quad (3\text{-}4)$$

Where:

$N_{SPF\,rs}$ = estimate of predicted average crash frequency for SPF base conditions for a rural two-lane two-way roadway segment (described in Section 10.6) (crashes/year);

$AADT$ = average annual daily traffic volume (vehicles per day) on roadway segment;

L = length of roadway segment (miles).

Applying Multiplicative Crash Modification Factors

Example 1
Treatment '*x*' consists of providing a left-turn lane on both major-road approaches to an urban four-leg signalized intersection, and treatment '*y*' is permitting right-turn-on-red maneuvers. These treatments are to be implemented, and it is assumed that their effects are independent of each other. An urban four-leg signalized intersection is expected to have 7.9 crashes/year. For treatment t_x, CMF_x = 0.81; for treatment t_y, CMF_y = 1.07.

What crash frequency is to be expected if treatment *x* and *y* are both implemented?

Answer to Example 1
Using Equation 3-3 and a calibration factor = 1.0, expected crashes = 7.9 × 0.81 × 1.07 = 6.8 crashes/year.

Example 2
The CMF for single-vehicle run-off-the-road crashes for a 1 percent increase in grade is 1.04 regardless of whether the increase is from 1 percent to 2 percent or from 5 percent to 6 percent. What is the effect of increasing the grade from 2 percent to 4 percent?

Answer to Example 2
Using Equation 3-7, expected single-vehicle run-off-the-road crashes will increase by a factor of $1.04^{(4-2)} = 1.04^2 = 1.08$ = 8 percent increase.

Multiplication of CMFs in Part C

In the Part C predictive method, an SPF estimate is multiplied by a series of CMFs to adjust the estimate of crash frequency from the base condition to the specific conditions present at a site. The CMFs are multiplicative because the effects of the features they represent are presumed to be independent. However, little research exists regarding the independence of these effects, but this is a reasonable assumption based on current knowledge. The use of observed crash frequency data in the EB Method can help to compensate for bias caused by lack of independence of the CMFs. As new research is completed, future HSM editions may be able to address the independence (or lack of independence) of these effects more fully.

Multiplication of CMFs in Part D

CMFs are also used in estimating the anticipated effects of proposed future treatments or countermeasures (e.g., in some of the methods discussed in Section C.8). The limited understanding of interrelationships between the various treatments presented in Part D requires consideration, especially when more than three CMFs are proposed. If CMFs are multiplied together, it is possible to overestimate the combined effect of multiple treatments when it is expected that more than one of the treatments may affect the same type of crash. The implementation of wider lanes and wider shoulders along a corridor is an example of a combined treatment where the independence of the individual treatments is unclear, because both treatments are expected to reduce the same crash types. When CMFs are multiplied, the practitioner accepts the assumption that the effects represented by the CMFs are independent of one another. Users should exercise engineering judgment to assess the interrelationship or independence, or both, of individual elements or treatments being considered for implementation.

Compatibility of Multiple CMFs

Engineering judgment is also necessary in the use of combined CMFs where multiple treatments change the overall nature or character of the site; in this case, certain CMFs used in the analysis of the existing site conditions and the proposed treatment may not be compatible. An example of this concern is the installation of a roundabout at an urban two-way stop-controlled or signalized intersection. The procedure for estimating the crash frequency after installation of a roundabout (see Chapter 12) is to estimate the average crash frequency for the existing site conditions (as an SPF for roundabouts in currently unavailable), and then apply a CMF for a conventional intersection to roundabout conversion. Installing a roundabout changes the nature of the site so that other CMFs applicable to existing urban two-way stop-controlled or signalized intersections may no longer be relevant.

Estimating Average Crash Frequency without Assuming Similar Crash Frequency in All Periods
This estimation of the average crash frequency of a specific roadway or facility in a certain period is conducted using crash counts from other periods without assuming that the expected average crash frequency of a specific roadway or facility's expected average crash frequency is similar in all periods. Equation 3A-5 presents the relationship that estimates a specific unit for the last period of a sequence.

$$\hat{\mu}_Y = \sum_{y=1}^{Y} X_y \bigg/ \sum_{y=1}^{Y} d_y \qquad (3A\text{-}5)$$

Where:

$\hat{\mu}_Y$ = most likely estimate of μ_Y (last period or year);

μ_y ≡ $\mu_y \times d_y$ where y denotes a period or a year (y=1, 2,…., Y; while Y denotes the last period or last year); e.g., for first period d_1 = relationship of μ_1/μ_Y;

X_y = the counts of crashes for each period or Year y.

Equation 3A-6 presents the estimate of the variance of $\hat{\mu}_Y$.

$$\hat{V}(\hat{\mu}_Y) = \sum_{y=1}^{Y} X_y \bigg/ \left(\sum_{y=1}^{Y} d_y\right)^2 \qquad (3A\text{-}6)$$

Where:

$\hat{\mu}_Y$ = most likely estimate of μ_Y (last period or year);

d_y = the μ_1/μ

X_y = the counts of crashes for each period or Year y.

For this estimate, it is necessary to add all crash counts reported during this year for all intersections that are similar to the intersection, under evaluation, throughout the network. Using the example given in Figure 3A-1 to illustrate this estimate, the proportion of the crashes counts per year in relation to the annual total crash counts for all similar intersections was calculated. The results are shown in Table 3A-2, e.g., 27 percent of annual crashes occur in the first year, 22 percent in the second year, etc.

Each yearly proportion is modified in relation to the last year, e.g., $d_1 = \mu_1/\mu_4 = 0.27/0.31 = 0.87$, as shown in Table 3A-2.

Table 3A-2. Illustration of Yearly Proportions and Relative Last Year Rates

	Year 1	Year 2	Year 3	Year 4 = *Y*
Proportion of Crashes	0.27	0.22	0.20	0.31
d_y (relative to the last year)	0.87	0.71	0.64	1

For each year, the crashes counts are 5, 7, 11, and 9, see Figure 3A-1. Using Equations 3A-5 and 3A-6:

$\hat{\mu}_{\text{Year 4}} = (5 + 7 + 11 + 9)/(0.87 + 0.71 + 0.64 + 1) = 32/3.22 = 9.94$ estimate of crashes for the last year:

$\hat{\sigma} = \sqrt{32/3.22^2} = \pm 1.8$ crashes as the standard error of the last year's estimate

Failure to account for the effects of RTM introduces the potential for "RTM bias", also known as "selection bias". RTM bias occurs when sites are selected for treatment based on short-term trends in observed crash frequency. For example, a site is selected for treatment based on a high observed crash frequency during a very short period of time (e.g., two years). However, the site's long-term crash frequency may actually be substantially lower and therefore the treatment may have been more cost-effective at an alternate site.

Performance Threshold

A performance threshold value provides a reference point for comparison of performance measure scores within a reference population. Sites can be grouped based on whether the estimated performance measure score for each site is greater than or less than the threshold value. Those sites with a performance measure score less than the threshold value can be studied in further detail to determine if reduction in crash frequency or severity is possible.

The method for determining a threshold performance value is dependent on the performance measure selected. The threshold performance value can be a subjectively assumed value, or calculated as part of the performance measure methodology. For example, threshold values are estimated based on: the average of the observed crash frequency for the reference population, an appropriate safety performance function, or Empirical Bayes methods. Table 4-2 summarizes whether or not each of the performance measures accounts for regression-to-the-mean bias or estimates a performance threshold, or both. The performance measures are presented in relative order of complexity, from least to most complex. Typically, the methods that require more data and address RTM bias produce more reliable performance threshold values.

Table 4-2. Stability of Performance Measures

Performance Measure	Accounts for RTM Bias	Method Estimates a Performance Threshold
Average Crash Frequency	No	No
Crash Rate	No	No
Equivalent Property Damage Only (EPDO) Average Crash Frequency	No	No
Relative Severity Index	No	Yes
Critical Rate	Considers data variance but does not account for RTM bias	Yes
Excess Predicted Average Crash Frequency Using Method of Moments	Considers data variance but does not account for RTM bias	Yes
Level of Service of Safety	Considers data variance but does not account for RTM bias	Expected average crash frequency plus/minus 1.5 standard deviations
Excess Expected Average Crash Frequency Using SPFs	No	Predicted average crash frequency at the site
Probability of Specific Crash Types Exceeding Threshold Proportion	Considers data variance; not effected by RTM Bias	Yes
Excess Proportions of Specific Crash Types	Considers data variance; not effected by RTM Bias	Yes
Expected Average Crash Frequency with EB Adjustments	Yes	Expected average crash frequency per year at the site
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment	Yes	Expected average crash frequency per year at the site
Excess Expected Average Crash Frequency with EB Adjustments	Yes	Expected average crash frequency per year at the site

$$Coefficient\ of\ Variation\ (CV) = \frac{\sqrt{Var(Performance\ Measure)}}{Performance\ Measure} \quad (4\text{-}1)$$

A large CV indicates a low level of precision in the estimate, and a small CV indicates a high level of precision in the estimate. The calculated CV is compared to a specified limiting CV. If the calculated CV is less than or equal to the CV limiting value, the performance measure meets the desired precision level, and the performance measure for a given window can potentially be considered for use in ranking the segment. If the calculated CV is greater than the CV limiting value, the window is automatically removed from further consideration in potentially ranking the segment based upon the value of the performance measure.

There is no specific CV value that is appropriate for all network screening applications. However, by adjusting the CV value the user can vary the number of sites identified by network screening as candidates for further investigation. An appropriate initial or default value for the CV is 0.5.

Peak Searching Method

Question

Segment B, in an urban four-lane divided arterial reference population, will be screened using the Excess Expected Average Crash Frequency performance measure. Segment B is 0.47 mi long. The CV limiting value is assumed to be 0.25. If the peak searching method is used to study this segment, how is the methodology applied and how is the segment potentially ranked relative to other sites considered in the screening?

Answer

Iteration #1

The following table shows the results of the first iteration. In the first iteration, the site is divided into 0.1-mi windows. For each window, the performance measure is calculated along with the CV.

The variance is given as:

$$VAR_B = \frac{(5.2-5.7)^2 + (7.8-5.7)^2 + (1.1-5.7)^2 + (6.5-5.7)^2 + (7.8-5.7)^2}{(5-1)} = 7.7$$

The Coefficient of Variation for Segment B1 is calculated using Equation 4-1 as shown below:

$$CV_{B1} = \frac{\sqrt{7.7}}{5.2} = 0.53$$

Example Application of Expected Average Crash Frequency with Empirical Bayes Adjustment (Iteration #1)

Subsegment	Window Position	Excess Expected Average Crash Frequency	Coefficient of Variation (CV)
B1	0.00 to 0.10 mi	5.2	0.53
B2	0.10 to 0.20 mi	7.8	0.36
B3	0.20 to 0.30 mi	1.1	2.53
B4	0.30 to 0.40 mi	6.5	0.43
B5	0.37 to 0.47 mi	7.8	0.36
	Average	5.7	—

Because none of the calculated CVs are less than the CV limiting value, none of the windows meet the screening criterion, so a second iteration of the calculations is required.

STEP 4—Rank Locations and Compare ***Relative Severity Index (RSI)***

1 2 3 4

The average RSI costs are calculated by dividing the RSI crash cost for each intersection by the number of crashes for the same intersection. The average RSI cost per intersection is also compared to the average RSI cost for its respective population.

The following table shows the intersection ranking for all 20 intersections based on their average RSI costs. The RSI costs for Intersection 7 would be compared to the average RSI cost for the unsignalized intersection population. In this instance, the average RSI cost for Intersection 7 ($31,700) is less than the average RSI cost for all unsignalized intersections ($39,700 from calculations in Step 3).

Ranking Based on Average RSI Cost per Intersection

Intersection	Average RSI Cost[a]	Exceeds RSI_p
2	$57,600	X
14	$52,400	X
6	$42,800	X
9	$44,100	X
20	$43,100	X
3	$42,400	X
4	$42,000	X
12	$41,000	X
11	$39,900	X
16	$39,500	
19	$37,800	
1	$37,400	
13	$34,800	
8	$34,600	
18	$34,100	
17	$32,900	
7	$31,700	
5	$31,400	
10	$31,000	
15	$30,600	

[a]Average RSI Costs per Intersection are rounded to the nearest $100.

4.4.2.5. Critical Rate

The observed crash rate at each site is compared to a calculated critical crash rate that is unique to each site. Sites that exceed their respective critical rate are flagged for further review. The critical crash rate depends on the average crash rate at similar sites, traffic volume, and a statistical constant that represents a desired confidence level.

Data Needs

- Crashes by location
- Traffic Volume

STEP 3—Calculate Crash Frequency Variance per Reference Population
Excess Predicted Average Crash Frequency Using Method of Moments

1 2 **3** 4 5 6

Use Equation 4-13 to calculate variance. Alternatively, variance can be more easily calculated with common spreadsheet programs.

$$Var(N) = \frac{\sum_{i=1}^{n}\left(N_{\text{observed},i} - N_{\text{observed},rp}\right)^{(2)}}{n_{\text{sites}} - 1} \quad (4\text{-}13)$$

Where:

$Var(N)$ = Variance

$N_{observed,rp}$ = Average crash frequency, per reference population

$N_{\text{observed},i}$ = Observed crash frequency per year at site i

n_{sites} = Number of sites per reference population

Calculate the crash frequency variance calculation for the TWSC reference population:

$$S_{TWSC}{}^{2} = \frac{112.8}{6} = 18.8$$

The variance for signal and TWSC reference populations is shown in the following table:

Reference Population	Crash Frequency	
	Average	Variance
Signal	6.1	13.75
TWSC	7.1	10.5

STEP 4—Calculate Adjusted Observed Crash Frequency per Site
Excess Predicted Average Crash Frequency Using Method of Moments

1 2 3 **4** 5 6

Using the variance and average crash frequency for a reference population, find the adjusted observed crash frequency for each site using Equation 4-14.

$$N_{\text{observed},i(adj)} = N_{\text{observed},i} + \frac{N_{\text{observed},rp}}{Var(N)} \times (N_{\text{observed},rp} - N_{\text{observed},i}) \quad (4\text{-}14)$$

STEP 1—Estimate Predicted Average Crash Frequency Using an SPF ***Level of Service of Safety (LOSS)***

1 2 3 4 5

Use the predictive method and SPFs outlined in Part C to estimate the average crash frequency. The predicted average crash frequency is summarized in Table 4-10:

Table 4-10. Estimated Predicted Average Crash Frequency from an SPF

Intersection	Year	AADT Major Street	AADT Minor Street	Predicted Average Crash Frequency from an SPF	Average 3-Year Predicted Crash Frequency from an SPF
	1	12,000	1,200	1.7	
2	2	12,200	1,200	1.7	1.7
	3	12,900	1,300	1.8	
	1	18,000	800	2.1	
3	2	18,900	800	2.2	2.2
	3	19,100	800	2.2	
	1	21,000	1,000	2.5	
7	2	21,400	1,000	2.5	2.6
	3	22,500	1,100	2.7	
	1	15,000	1,500	2.1	
10	2	15,800	1,600	2.2	2.2
	3	15,900	1,600	2.2	
	1	26,000	500	2.5	
15	2	26,500	300	2.2	2.3
	3	27,800	200	2.1	
	1	14,400	3,200	2.5	
17	2	15,100	3,400	2.6	2.6
	3	15,300	3,400	2.6	
	1	15,400	2,500	2.4	
19	2	15,700	2,500	2.5	2.5
	3	16,500	2,600	2.6	

STEP 2—Calculate Standard Deviation ***Level of Service of Safety (LOSS)***

1 2 3 4 5

Calculate the standard deviation of the predicted crashes. Equation 4-16 is used to calculate the standard deviation. This estimate of standard deviation is valid since the SPF assumes a negative binomial distribution of crash counts.

$$\sigma = \sqrt{kN^2_{predicted}} \tag{4-16}$$

Where:

σ = Standard deviation

k = Overdispersion parameter of the SPF

$N_{predicted}$ = Predicted average crash frequency from the SPF

As shown, the standard deviation calculations for Intersection 7 are

$$\sigma = \sqrt{0.40 \times 2.6^2} = 1.6$$

The standard deviation calculation is performed for each intersection. The standard deviation for the TWSC intersections is summarized in the following table:

Intersection	Average Observed Crash Frequency	Predicted Average Crash Frequency from an SPF	Standard Deviation
2	11.7	1.7	1.1
3	7.7	2.2	1.4
7	11.3	2.6	1.6
10	5.7	2.2	1.4
15	5.7	2.3	1.5
17	4.3	2.6	1.6
19	3.7	2.5	1.6

STEP 3—Calculate Limits for LOSS Categories ***Level of Service of Safety (LOSS)***

1 2 **3** 4 5

Calculate the limits for the four LOSS categories for each intersection using the equations summarized in Table 4-11.

Table 4-11. LOSS Categories

LOSS	Condition	Description
I	$\sigma < N_{observed} < (N_{predicted} - 1.5 \times (\sigma))$	Indicates a low potential for crash reduction
II	$(N_{predicted} - 1.5 \times (\sigma)) \leq N_{observed} < N_{predicted}$	Indicates low to moderate potential for crash reduction
III	$N_{predicted} \leq N_{observed} < (N_{predicted} + 1.5 \times (\sigma))$	Indicates moderate to high potential for crash reduction
IV	$N_{observed} \geq (N_{predicted} + 1.5 \times (\sigma))$	Indicates a high potential for crash reduction

This sample calculation for Intersection 7 demonstrates the upper limit calculation for LOSS III.

$$N_{predicted} + 1.5 \times (\sigma) = 2.6 + 1.5 \times (1.6) = 5.0$$

A similar pattern is followed for the other LOSS limits.

The values for this calculation are provided in the following table:

LOSS Limits for Intersection 7

Intersection	LOSS I Limits	LOSS II Limits	LOSS III Upper Limit	LOSS IV Limits
7	0 to 0.2	0.2 to 2.6	2.6 to 5.0	≥ 5.0

STEP 4—Calculate Alpha and Beta Parameters	*Probability of Specific Crash Types Exceeding Threshold Proportion*

1 2 3 **4** 5 6

Calculate the sample mean proportion of target crashes by type or severity for all sites under consideration using Equation 4-21.

$$\overline{p_i^*} = \frac{\sum p_i}{n_{\text{sites}}}, N_{\text{observed},i} \geq 2 \tag{4-21}$$

Where:

n_{sites} = Total number of sites being analyzed

$\overline{p^*_i}$ = Mean proportion of target crash types

p_i = Observed proportion

Calculate Alpha (α) and Beta (ß) for each subcategory using Equations 4-22 and 4-23.

$$\alpha = \frac{\overline{p^*_i}^2 - \overline{p^*_i}^3 - s^2(\overline{p^*_i})}{Var(N)} \tag{4-22}$$

$$\beta = \frac{\alpha}{\overline{p^*_i}} - \alpha \tag{4-23}$$

Where:

$Var(N)$ = Variance (equivalent to the square of the standard deviation, s^2)

$\overline{p^*_i}$ = Mean proportion of target crash types

The calculation for the two-way stop-controlled subcategory is:

$$\alpha = \frac{0.22^2 - 0.22^3 - 0.037 \times 0.22}{0.037} = 0.80$$

$$\beta = \frac{0.80}{0.22} - 0.80 = 2.84$$

The following table shows the numerical values used in the equations and summarizes the alpha and beta calculations for the TWSC intersections:

Alpha and Beta Calculations

Subcategories	s^2	$\overline{p^*_i}$	α	ß
TWSC	0.037	0.22	0.80	2.84

The TWSC intersection population is ranked based on the Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure as shown in the following table:

Ranking Based on Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure

Intersections	Probability
2	1.00
11	0.98
9	0.83
12	0.75
16	0.48
6	0.48
13	0.48
20	0.41
4	0.35
17	0.25
5	0.21
1	0.19
18	0.19
7	0.13
10	0.13
3	0.04

4.4.2.10 Excess Proportion of Specific Crash Types

Sites are evaluated to quantify the extent to which a specific crash type is overrepresented compared to other crash types at a location. The sites are ranked based on excess proportion, which is the difference between the true proportion, p_i, and the threshold proportion, $p*_i$. The excess is calculated for a site if the probability that a site's long-term observed proportion is higher than the threshold proportion, $p*_i$, exceeds a certain limiting probability (e.g., 90 percent).

Data Needs

- Crash data by type and location

Strengths and Limitations

The strengths and limitations of the Excess Proportions of Specific Crash Types Proportion performance measure include the following:

Strengths	Limitations
Can also be used as a diagnostic tool	Does not account for traffic volume.
Considers variance in data	Some sites may be identified for further study because of unusually low frequency of non-target crash types
Not effected by RTM Bias	

Procedure

Calculation of the excess proportion follows the same procedure outlined in Steps 1 through 5 of the Probability of Specific Crash Types Exceeding Threshold Proportions method. Therefore, the procedure outlined in this section builds on the previous method and applies results of sample calculations shown above in the example table of Step 6.

STEP 2—Calculate Annual Correction Factor

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

1 **2** 3 4 5 6 7

Calculate the annual correction factor (C_n) at each intersection for each year and each severity (i.e., total and *FI*).

The annual correction factor is predicted average crash frequency from an SPF for year n divided by the predicted average crash frequency from an SPF for year 1. This factor is intended to capture the effect that annual variations in traffic, weather, and vehicle mix have on crash occurrences. (3)

$$C_{n(\text{total})} = \frac{N_{\text{predicted},n(\text{total})}}{N_{\text{predicted},1(\text{total})}} \quad \text{and} \quad C_{n(FI)} = \frac{N_{\text{predicted},n(FI)}}{N_{\text{predicted},1(FI)}} \tag{4-26}$$

Where:

$C_{n(\text{total})}$ = Annual correction factor for total crashes

$C_{n(FI)}$ = Annual correction factor for fatal or injury crashes, or both

$N_{\text{predicted},\,n(\text{total})}$ = Predicted number of total crashes for year *n*

$N_{\text{predicted},n(FI)}$ = Predicted number of fatal or injury crashes, or both, for year *n*

Shown below is the calculation for Intersection 7 based on the annual correction factor for year 3. The predicted crashes shown in the equation are the result of Step 1 and are summarized in the table that follows.

$$C_{3(\text{total})} = \frac{2.7}{2.5} = 1.1$$

$$C_{3(FI)} = \frac{1.1}{1.0} = 1.1$$

This calculation is repeated for each year and each intersection. The following table summarizes the annual correction factor calculations for the TWSC intersections:

7.4.3.4. Method Two—Convert Non-Uniform Annual Benefits to Present Value

Some countermeasures yield larger changes in expected average crash frequency in the first years after implementation than in subsequent years. In order to account for this occurrence over the service life of the countermeasure, non-uniform annual monetary values can be calculated as shown in Step 1 below for each year of service. The following process is used to convert the project benefits of all non-uniform annual monetary values to a single present value:

1. Convert each annual monetary value to its individual present value. Each future annual value is treated as a single future value; therefore, a different present worth factor is applied to each year.

    a) Substitute the $(P/F,i,y)$ factor calculated for each year in the service life for the $(P/A,i,y)$ factor presented in Equation 7-2.

        i) $(P/F,i,y)$ = a factor that converts a single future value to its present value

        ii) $(P/F,i,y) = (1+i)^{(-y)}$

        Where:

        i = discount rate (i.e., the discount rate is 4 percent, $i = 0.04$)

        y = year in the service life of the countermeasure(s)

2. Sum the individual present values to arrive at a single present value that represents the project benefits of the project.

The sample problems at the end of this chapter illustrate how to convert non-uniform annual values to a single present value.

7.5. ESTIMATE PROJECT COSTS

Estimating the costs associated with implementing a countermeasure follows the same procedure as performing cost estimates for other construction or program implementation projects. Similar to other roadway improvement projects, expected project costs are unique to each site and to each proposed countermeasure(s). The cost of implementing a countermeasure or set of countermeasures could include a variety of factors, e.g., right-of-way acquisition, construction material costs, grading and earthwork, utility relocation, environmental impacts, maintenance, and other costs, including any planning and engineering design work conducted prior to construction.

The AASHTO Redbook states, "Project costs should include the present value of any obligation to incur costs (or commit to incur costs in the future) that burden the [highway] authority's funds." (1) Therefore, under this definition the present value of construction, operating, and maintenance costs over the service life of the project are included in the assessment of expected project costs. Chapter 6 of the AASHTO Redbook provides additional guidance regarding the categories of costs and their proper treatment in a benefit-cost or economic appraisal. Categories discussed in the Redbook include:

- Construction and other development costs
- Adjusting development and operating cost estimates for inflation
- The cost of right-of-way
- Measuring the current and future value of undeveloped land
- Measuring current and future value of developed land
- Valuing already-owned right-of-way
- Maintenance and operating costs
- Creating operating cost estimates

Table 7-3. Expected Average Crash Frequency at Intersection 2 WITHOUT Installing the Roundabout

Year in service life (y)	Major AADT	Minor AADT	$N_{expected(total)}$	$N_{expected(FI)}$
1	23,553	1,758	10.4	5.2
2	23,906	1,785	10.5	5.3
3	24,265	1,812	10.5	5.3
4	24,629	1,839	10.6	5.4
5	24,998	1,866	10.7	5.4
6	25,373	1,894	10.7	5.4
7	25,754	1,923	10.8	5.5
8	26,140	1,952	10.9	5.5
9	26,532	1,981	11.0	5.5
10	26,930	2,011	11.0	5.6
Total			107.1	54.1

The roadway agency finds the societal crash costs shown in Table 7-4 acceptable. The agency decided to conservatively estimate the economic benefits of the countermeasures. Therefore, they are using the average injury crash cost (i.e., the average value of a fatal (K), disabling (A), evident (B), and possible injury crash (C) as the crash cost value representative of the predicted fatal and injury crashes.

Table 7-4. Societal Crash Costs by Severity

Injury Severity	Estimated Cost
Fatal (K)	$4,008,900
Cost for crashes with a fatal and/or injury (K/A/B/C)	$158,200
Disabling Injury (A)	$216,000
Evident Injury (B)	$79,000
Possible Injury (C)	$44,900
PDO (O)	$7,400

Source: *Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries*, FHWA-HRT-05-051, October 2005

Assumptions regarding the service life for the roundabout, the annual traffic growth at the site during the service life, the discount rate and the cost of implementing the roundabout include the following:

	Intersection 2
Countermeasure	Roundabout
Service Life	10 years
Annual Traffic Growth	2%
Discount Rate (i)	4.0%
Cost Estimate Method	$695,000

The following steps are required to solve the problem.

- *Step 1*—Calculate the expected average crash frequency at Intersection 2 without the roundabout.
- *Step 2*—Calculate the expected average crash frequency at Intersection 2 with the roundabout.
- *Step 3*—Calculate the change in expected average crash frequency for total, fatal and injury, and PDO crashes.
- *Step 4*—Convert the change in crashes to a monetary value for each year of the service life.
- *Step 5*—Convert the annual monetary values to a single present value representative of the total monetary benefits expected from installing the countermeasure at Intersection 2.

A summary of inputs, equations, and results of economic appraisal conducted for Intersection 2 is shown in Table 7-5. The methods for conducting the appraisal are outlined in detail in the following sections.

Step 3—Calculate the expected change in crash frequency for total, fatal and injury, and PDO crashes.
The difference between the expected average crash frequency with and without the countermeasure is the expected change in average crash frequency. Equations 7-8, 7-9, and 7-10 are used to estimate this change for total, fatal and injury, and PDO crashes.

$$\Delta N_{\text{expected}(FI)} = N_{\text{expected}(FI)} - N_{\text{expected roundabout}(FI)} \quad (7\text{-}8)$$

$$\Delta N_{\text{expected(total)}} = N_{\text{expected(total)}} - N_{\text{expected roundabout(total)}} \quad (7\text{-}9)$$

$$\Delta N_{\text{expected}(PDO)} = \Delta N_{\text{expected(total)}} - \Delta N_{\text{expected}(FI)} \quad (7\text{-}10)$$

Where:

$\Delta N_{\text{expected(total)}}$ = Expected change in average crash frequency due to implementing countermeasure;

$\Delta N_{\text{expected}(FI)}$ = Expected change in average fatal and injury crash frequency due to implementing countermeasure; and

$\Delta N_{\text{expected}(PDO)}$ = Expected change in average PDO crash frequency due to implementing countermeasure.

Table 7-8 summarizes the expected change in average crash frequency due to installing the roundabout.

Table 7-8. Change in Expected Average in Crash Frequency at Intersection 2 WITH the Roundabout

Year in service life, *y*	$\Delta N_{\text{expected(total)}}$	$\Delta N_{\text{expected}(FI)}$	$\Delta N_{\text{expected}(PDO)}$
1	4.6	4.3	0.3
2	4.6	4.3	0.3
3	4.6	4.3	0.3
4	4.7	4.4	0.3
5	4.7	4.4	0.3
6	4.7	4.4	0.3
7	4.8	4.5	0.3
8	4.8	4.5	0.3
9	4.8	4.5	0.3
10	4.8	4.6	0.2
Total	47.1	44.2	2.9

Step 4—Convert Change in Crashes to a Monetary Value
The estimated reduction in average crash frequency can be converted to a monetary value for each year of the service life using Equations 7-11 through 7-13.

$$AM_{(PDO)} = \Delta N_{\text{expected}(PDO)} \times CC_{(PDO)} \quad (7\text{-}11)$$

$$AM_{(FI)} = \Delta N_{\text{expected}(FI)} \times CC_{(FI)} \quad (7\text{-}12)$$

$$AM_{\text{(total)}} = AM_{(PDO)} + AM_{(FI)} \quad (7\text{-}13)$$

Where:

$AM_{(PDO)}$ = Monetary value of the estimated change in average PDO crash frequency for year, *y*;

$CC_{(PDO)}$ = Crash cost for PDO crash severity;

$AM_{(FI)}$ = Monetary value of the estimated change in fatal and injury average crash frequency for year *y*;

$CC_{(FI)}$ = Crash cost for FI crash severity; and

$AM_{\text{(total)}}$ = Monetary value of the total estimated change in average crash frequency for year *y*.

Table 7-9 summarizes the monetary value calculations for each year of the service life.

Table 7-9. Annual Monetary Value of Change in Crashes

Year in service life, *y*	$\Delta N_{(FI)}$	FI Crash Cost	$AM_{(FI)}$	$\Delta N_{(PDO)}$	PDO Crash Cost	$AM_{(PDO)}$	$AM_{(total)}$
1	4.3	$158,200	$680,260	0.3	$7,400	$2,220	$682,480
2	4.3	$158,200	$680,260	0.3	$7,400	$2,220	$682,480
3	4.3	$158,200	$680,260	0.3	$7,400	$2,220	$682,480
4	4.4	$158,200	$696,080	0.3	$7,400	$2,220	$698,300
5	4.4	$158,200	$696,080	0.3	$7,400	$2,220	$698,300
6	4.4	$158,200	$696,080	0.3	$7,400	$2,220	$698,300
7	4.5	$158,200	$711,900	0.3	$7,400	$2,220	$714,120
8	4.5	$158,200	$711,900	0.3	$7,400	$2,220	$714,120
9	4.5	$158,200	$711,900	0.3	$7,400	$2,220	$714,120
10	4.6	$158,200	$727,720	0.2	$7,400	$1,480	$729,200

Step 5—Convert Annual Monetary Values to a Present Value

The total monetary benefits expected from installing a roundabout at Intersection 2 are calculated as a present value using Equations 7-14 and 7-15.

Note—A 4 percent discount rate is assumed for the conversion of the annual values to a present value.

Convert the annual monetary value to a present value for each year of the service life.

$$PV_{\text{benefits}} = \text{Total Annual Monetary Benefits} \times (P/F,i,y) \tag{7-14}$$

Where:

PV_{benefits} = Present value of the project benefits per site in year y;

$(P/F,i,y)$ = Factor that converts a single future value to its present value, calculated as $(1+i)^{(-y)}$;

i = Discount rate (i.e., the discount rate is 4 percent, $i = 0.04$); and

y = Year in the service life of the countermeasure.

If the annual project benefits are uniform, then the following factor is used to convert a uniform series to a single present worth:

$$(P/A,i,y) = \frac{(1.0 + i)^{(y)} - 1.0}{i \times (1.0 + i)^{(y)}} \tag{7-15}$$

Where:

$(P/A,i,y)$ = factor that converts a series of uniform future values to a single present value.

Table 7-10 summarizes the results of converting the annual values to present values.

Table 7-10. Converting Annual Values to Present Values

Year in service life (*y*)	(*P/F,i,y*)	$AM_{(total)}$	Present Value
1	0.96	$682,480	$656,230
2	0.92	$682,480	$630,990
3	0.89	$682,480	$606,720
4	0.85	$698,300	$596,910
5	0.82	$698,300	$573,950
6	0.79	$698,300	$551,880
7	0.76	$714,120	$542,670
8	0.73	$714,120	$521,800
9	0.70	$714,120	$501,730
10	0.68	$729,200	$492,620
Total			$5,675,500

The total present value of the benefits of installing a roundabout at Intersection 2 is the sum of the present value for each year of the service life. The sum is shown above in Table 7-10.

Results

The estimated present value monetary benefit of installing a roundabout at Intersection 2 is $5,675,500.

The roadway agency estimates the cost of installing the roundabout at Intersection 2 is $2,000,000.

If this analysis were intended to determine whether the project is cost-effective, the magnitude of the monetary benefit provides support for the project. If the monetary benefit of change in crashes at this site were to be compared to other sites the BCR could be calculated and used to compare this project to other projects in order to identify the most economically efficient project.

7.10. REFERENCES

(1) AASHTO. *A Manual of User Benefit Analysis for Highways*, 2nd Edition. American Association of State Highway and Transportation Officials, Washington, DC, 2003.

(2) Council, F. M., E. Zaloshnja, T. Miller, and B. Persaud. *Crash Cost Estimates by Maximum Police Reported Injury Severity within Selected Crash Geometries*. Publication No. FHWA-HRT-05-051. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, October 2005.

(3) Harwood, D. W. et al. *Safety Analyst: Software Tools for Safety Management of Specific Highway Sites Task M Functional Specification for Module 3*. Economic Appraisal and Priority Ranking GSA Contract No. GS-23F-0379K Task No. DTFH61-01-F-00096. Midwest Research Institute for FHWA. November 2003. More information available from http://www.safetyanalyst.org.

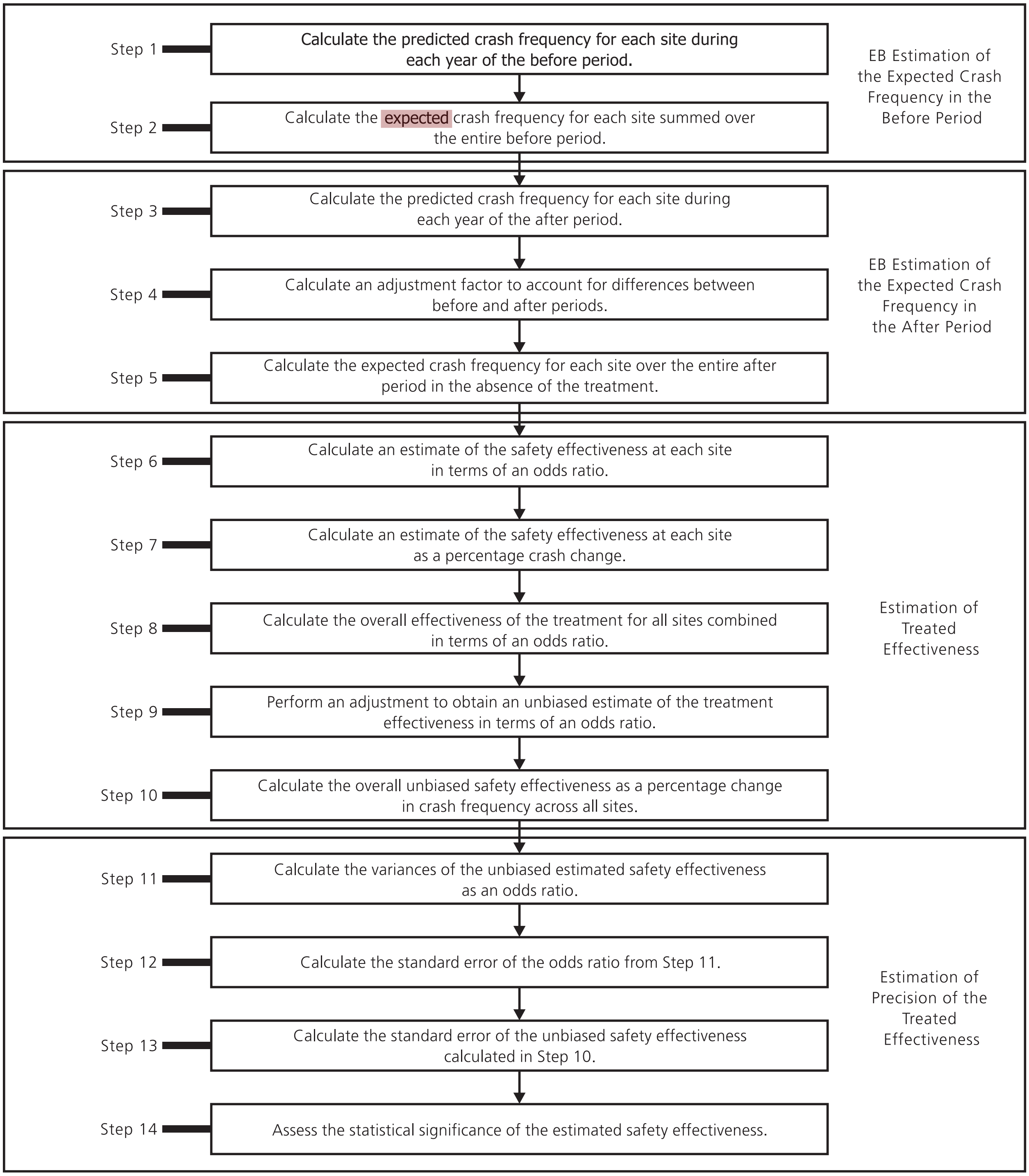


Figure 9-2. Overview of EB Before/After Safety Evaluation

Step 4—Calculate the Adjustment Factor, *r*, to account for the differences between the before and after periods in duration and traffic volume at each site.
Using Equation 9A.1-3 and Columns 25 and 19, calculate the adjustment factor, *r*, to account for the differences between the before and after periods in duration and traffic volume at each site. The results appear in Column 26 in the table presented in Step 3.

Step 5—Calculate the Expected Average Crash Frequency for each Site over the Entire after Period in the Absence of the Treatment.
Using Equation 9A.1-4 and Columns 22 and 26, calculate the expected average crash frequency for each site over the entire after period in the absence of the treatment. The results appear in Column 27 in the table presented in Step 3.

9.10.4. Estimation of the Treatment Effectiveness

Step 6—Calculate an Estimate of the Safety Effectiveness of the Treatment at each site in the form of an odds ratio.
Using Equation 9A.1-5 and Columns 13 and 27, calculate an estimate of the safety effectiveness of the treatment at each site in the form of an odds ratio. The results appear in Column 28.

(1)	(13)	(27)	(28)	(29)	(30)
Site No.	Observed crash frequency in after period	Expected average crash frequency in after period without treatment	Odds ratio	Safety effectiveness (%)	Variance term (Eq. 9A.1-11)
1	2	6.08	0.329	67.13	1.787
2	2	3.02	0.662	33.84	0.939
3	2	1.87	1.068	–6.75	0.582
4	1	5.40	0.185	81.47	1.440
5	1	0.79	1.274	–27.35	0.209
6	1	1.89	0.530	46.96	0.499
7	9	5.61	1.604	–60.44	1.486
8	0	3.50	0.000	100.00	0.817
9	0	2.71	0.000	100.00	0.627
10	0	1.40	0.000	100.00	0.323
11	5	2.84	1.758	–75.81	0.657
12	6	3.17	1.894	–89.44	0.732
13	1	4.60	0.217	78.26	1.063
Total	30	42.88			11.162

Step 7—Calculate the Safety Effectiveness as a percentage crash change at each site.
Using Equation 9A.1-6 and Column 28, calculate the safety effectiveness as a percentage crash change at each site. The results appear in Column 29 in the table presented in Step 6. A positive result indicates a reduction in crashes; conversely, a negative result indicates an increase in crashes.

Step 8—Calculate the Overall Effectiveness of the Treatment for all sites combined, in the form of an odds ratio.
Using Equation 9A.1-7 and the totals from Columns 13 and 27 (Step 6), calculate the overall effectiveness of the treatment for all sites combined, in the form of an odds ratio:

$$OR' = \frac{30}{42.88} = 0.700$$

Step 5—Calculate the expected average crash frequency, N_{expected}, for each site *i*, over the entire after period in the absence of the treatment as:

$$N_{\text{expected},A} = N_{\text{expected},B} \times r_i \qquad (9A.1\text{-}4)$$

Estimation of Treatment Effectiveness

Step 6—Calculate an estimate of the safety effectiveness of the treatment at each site *i* in the form of an odds ratio, OR_i, as:

$$OR_i = \frac{N_{\text{observed},A}}{N_{\text{expected},A}} \qquad (9A.1\text{-}5)$$

Where:

OR_i = Odd ratio at site *i*

$N_{\text{observed},A}$ = Observed crash frequency at site *i* for the entire after period

Step 7—Calculate the safety effectiveness as a percentage crash change at site *i* as:

$$\text{Safety Effectiveness}_i = 100 \times (1 - OR_i) \qquad (9A.1\text{-}6)$$

Step 8—Calculate the overall effectiveness of the treatment for all sites combined, in the form of an odds ratio, OR', as follows:

$$OR' = \frac{\sum_{\text{All sites}} N_{\text{observed},A}}{\sum_{\text{All sites}} N_{\text{expected},A}} \qquad (9A.1\text{-}7)$$

Step 9—The odds ratio, OR', calculated in Equation 9A.1-7 is potentially biased; therefore, an adjustment is needed to obtain an unbiased estimate of the treatment effectiveness in terms of an adjusted odds ratio, *OR*. This is calculated as follows:

$$OR = \frac{OR'}{1 + \dfrac{Var\left(\sum_{\text{All sites}} N_{\text{expected},A}\right)}{\left(\sum_{\text{All sites}} N_{\text{expected},A}\right)^2}} \qquad (9A.1\text{-}8)$$

Where:

$$Var\left(\sum_{\text{All sites}} N_{\text{expected},A}\right) = \sum_{\text{All sites}} \left[(r_i)^2 \times N_{\text{expected},B} \times \left(1 - w_{i,B}\right)\right] \qquad (9A.1\text{-}9)$$

and $w_{i,B}$ is defined in Equation 9A.1-2 and r_i is defined in Equation 9A.1-3.

Where:

$$R^2_{i(SE)} = \frac{1}{N_{\text{observed},T,B,\text{total}}} + \frac{1}{N_{\text{observed},T,A,\text{total}}} + \frac{1}{N_{\text{expected},C,B,\text{total}}} + \frac{1}{N_{\text{expected},C,A,\text{total}}} \quad (9A.2\text{-}13)$$

Step 12—Using Equation 9A.2-14, calculate the weighted average log odds ratio, *R*, across all *n* treatment sites as:

$$R = \frac{\sum_n w_i R_i}{\sum_n w_i} \quad (9A.2\text{-}14)$$

Step 13—Exponentiating the result from Equation 9A.2-14, calculate the overall effectiveness of the treatment, expressed as an odds ratio, *OR*, averaged across all sites, as follows:

$$OR = e^R \quad (9A.2\text{-}15)$$

Step 14—Calculate the overall safety effectiveness, expressed as a percentage change in crash frequency averaged across all sites as:

$$\text{Safety Effectiveness} = 100 \times (1 - OR) \quad (9A.2\text{-}16)$$

Step 15—To obtain a measure of the precision of the treatment effectiveness, calculate its standard error, *SE*(Safety Effectiveness), as follows:

$$SE(\text{Safety Effectiveness}) = 100 \frac{OR}{\sqrt{\sum_n w_i}} \quad (9A.2\text{-}17)$$

Step 16—Assess the statistical significance of the estimated safety effectiveness by making comparisons with the measure *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] and drawing conclusions based on the following criteria:

- If *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] < 1.7, conclude that the treatment effect is not significant at the (approximate) 90 percent confidence level.
- If *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] ≥ 1.7, conclude that the treatment effect is significant at the (approximate) 90 percent confidence level.
- If *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] ≥ 2.0, conclude that the treatment effect is significant at the (approximate) 95 percent confidence level.

9A.3 COMPUTATIONAL PROCEDURE FOR IMPLEMENTING THE SHIFT OF PROPORTIONS SAFETY EFFECTIVENESS EVALUATION METHOD

A computational procedure using the evaluation study method for assessing shifts in proportions of target collision types to determine the safety effectiveness of the treatment being evaluated, $AvgP_{(CT)diff}$, and to assess its statistical significance, follows.

This step-by-step procedure uses the same notation as that used in the traditional comparison-group safety evaluation method. All proportions of specific crash types (subscript "*CT*") are relative to total crashes (subscript "total").

Using these definitions, the roadway segment predictive models estimate the frequency of crashes that would occur on the roadway if no intersection were present. The intersection predictive models estimate the frequency of additional crashes that occur because of the presence of the intersection.

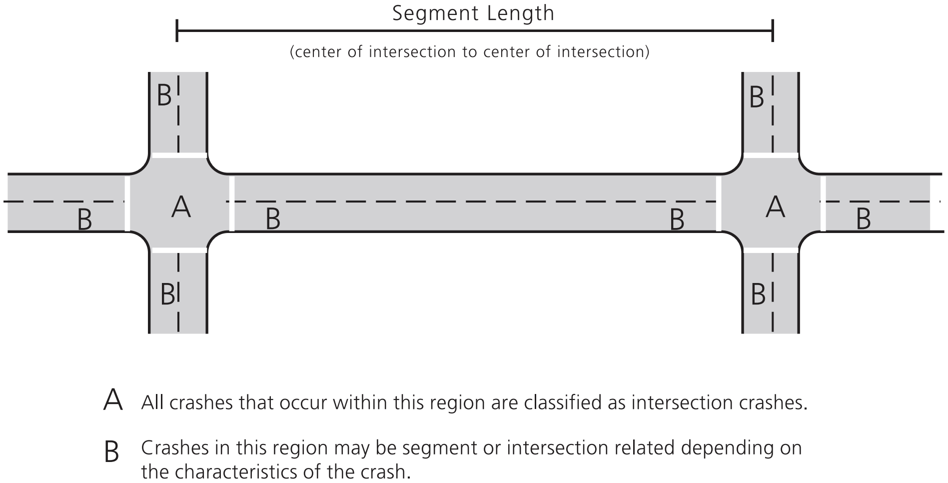


Figure C-3. Definition of Roadway Segments and Intersections

C.6.3 Safety Performance Functions (SPFs)

SPFs are regression models for estimating the predicted average crash frequency of individual roadway segments or intersections. In Step 9 of the predictive method, the appropriate SPFs are used to determine the predicted average crash frequency for the selected year for specific base conditions. Each SPF in the predictive method was developed with observed crash data for a set of similar sites. In the SPFs developed for the HSM, the dependent variable estimated is the predicted average crash frequency for a roadway segment or intersection under base conditions and the independent variables are the AADTs of the roadway segment or intersection legs (and, in some cases a few additional variables such as the length of the roadway segment).

An example of an SPF (for rural two-way two-lane roadway segments from Chapter 10) is shown in Equation C-4.

$$N_{spf\,rs} = (AADT) \times (L) \times (365) \times 10^{(-6)} \times e^{(-0.312)} \qquad \text{(C-4)}$$

Where:

$N_{spf\,rs}$ = predicted average crash frequency estimated for base conditions using a statistical regression model;

$AADT$ = annual average daily traffic volume (vehicles/day) on roadway segment; and

L = length of roadway segment (miles).

SPFs are developed through statistical multiple regression techniques using historic crash data collected over a number of years at sites with similar characteristics and covering a wide range of AADTs. The regression parameters of the SPFs are determined by assuming that crash frequencies follow a negative binomial distribution. The negative binomial distribution is an extension of the Poisson distribution which is typically used for crash frequencies. However, the mean and the variance of the Poisson distribution are equal. This is often not the case for crash frequencies where the variance typically exceeds the mean.

The negative binomial distribution incorporates an additional statistical parameter, the overdispersion parameter that is estimated along with the parameters of the regression equation. The overdispersion parameter has positive values. The greater the overdispersion parameter, the more that crash data vary as compared to a Poisson distribution with the same mean. The overdispersion parameter is used to determine a weighted adjustment factor for use in the EB Method described in Section C.6.6.

Crash modification factors (CMFs) are applied to the SPF estimate to account for geometric or geographic differences between the base conditions of the model and local conditions of the site under consideration. CMFs and their application to SPFs are described in Section C.6.4.

In order to apply an SPF, the following information relating to the site under consideration is necessary:

- Basic geometric design and geographic information of the site to determine the facility type and whether an SPF is available for that site type;
- AADT information for estimation of past periods, or forecast estimates of AADT for estimation of future periods; and
- Detailed geometric design of the site and base conditions (detailed in each of the Part C chapters) to determine whether the site conditions vary from the base conditions and therefore a CMF is applicable.

Updating Default Values of Crash Severity and Collision Type Distribution for Local Conditions

In addition to estimating the predicted average crash frequency for all crashes, SPFs can be used to estimate the distribution of crash frequency by crash severity types and by collision types (such as single-vehicle or driveway crashes). The distribution models in the HSM are default distributions.

Where sufficient and appropriate local data are available, the default values (for crash severity types and collision types and the proportion of night-time crashes) can be replaced with locally derived values when it is explicitly stated in Chapters 10, 11, and 12. Calibration of default distributions to local conditions is described in detail in Part C, Appendix A.1.1.

Development of Local SPFs

Some HSM users may prefer to develop SPFs with data from their own jurisdiction for use with the predictive method rather than calibrating the SPFs presented in the HSM. Part C, Appendix A provides guidance on developing jurisdiction-specific SPFs that are suitable for use with the predictive method. Development of jurisdiction-specific SPFs is not required.

C.6.4. Crash Modification Factors (CMFs)

In Step 10 of the predictive method, CMFs are determined and applied to the results of Step 9. The CMFs are used in Part C to adjust the predicted average crash frequency estimated by the SPF for a site with base conditions to the predicted average crash frequency for the specific conditions of the selected site.

CMFs are the ratio of the estimated average crash frequency of a site under two different conditions. Therefore, a CMF represents the relative change in estimated average crash frequency due to a change in one specific condition (when all other conditions and site characteristics remain constant).

Equation C-5 shows the calculation of a CMF for the change in estimated average crash frequency from site condition ‘a’ to site condition ‘b’.

$$CMF = \frac{\text{estimated average crash frequency with condition "b"}}{\text{estimated average crash frequency with condition "a"}} \tag{C-5}$$

CMFs defined in this way for expected crashes can also be applied to the comparison of predicted crashes between site condition ‘a’ and site condition ‘b’.

CMFs are an estimate of the effectiveness of the implementation of a particular treatment, also known as a countermeasure, intervention, action, or alternative design. Examples include: illuminating an unlighted road segment, paving gravel shoulders, signalizing a stop-controlled intersection, increasing the radius of a horizontal curve, or

C.6.5. Calibration of Safety Performance Functions to Local Conditions

The predictive models in Chapters 10, 11, and 12 have three basic elements: safety performance functions, crash modification factors, and a calibration factor. The SPFs were developed as part of HSM-related research from the most complete and consistent available data sets. However, the general level of crash frequencies may vary substantially from one jurisdiction to another for a variety of reasons including crash reporting thresholds and crash reporting system procedures. These variations may result in some jurisdictions experiencing substantially more reported traffic crashes on a particular facility type than in other jurisdictions. In addition, some jurisdictions may have substantial variations in conditions between areas within the jurisdiction (e.g., snowy winter driving conditions in one part of the state and only wet winter driving conditions in another part of the state). Therefore, for the predictive method to provide results that are reliable for each jurisdiction that uses them, it is important that the SPFs in Part C be calibrated for application in each jurisdiction. Methods for calculating calibration factors for roadway segments, C_r, and intersections, C_i, are included in Part C, Appendix A to allow highway agencies to adjust the SPF to match local conditions.

The calibration factors will have values greater than 1.0 for roadways that, on average, experience more crashes than the roadways used in developing the SPFs. Roadways that, on average, experience fewer crashes than the roadways used in the development of the SPF, will have calibration factors less than 1.0.

C.6.6. Weighting Using the Empirical Bayes Method

Step 13 or Step 15 of the predictive method are optional steps that are applicable only when observed crash data are available for either the specific site or the entire facility of interest. Where observed crash data and a predictive model are available, the reliability of the estimation is improved by combining both estimates. The predictive method in Part C uses the Empirical Bayes method, herein referred to as the EB Method.

The EB Method can be used to estimate expected average crash frequency for past and future periods and used at either the site-specific level or the project-specific level (where observed data may be known for a particular facility, but not at the site-specific level).

For an individual site (i.e., the site-specific EB Method) the EB Method combines the observed crash frequency with the predictive model estimate using Equation C-8. The EB Method uses a weighted factor, w, which is a function of the SPFs overdispersion parameter, k, to combine the two estimates. The weighted adjustment is therefore dependant only on the variance of the SPF model. The weighted adjustment factor, w, is calculated using Equation C-9.

$$N_{expected} = w \times N_{predicted} + (1.00 - w) \times N_{observed} \tag{C-8}$$

$$w = \frac{1}{1 + k \times \left(\sum_{\substack{\text{all study} \\ \text{years}}} N_{predicted} \right)} \tag{C-9}$$

Where:

$N_{expected}$ = estimate of expected average crash frequency for the study period;

$N_{predicted}$ = predictive model estimate of predicted average crash frequency for the study period;

$N_{observed}$ = observed crash frequency at the site over the study period;

w = weighted adjustment to be placed on the SPF prediction; and

k = overdispersion parameter from the associated SPF.

Table 10-3. Default Distribution for Crash Severity Level on Rural Two-Lane, Two-Way Roadway Segments

Crash Severity Level	Percentage of Total Roadway Segment Crashes[a]
Fatal	1.3
Incapacitating Injury	5.4
Nonincapacitating injury	10.9
Possible injury	14.5
Total fatal plus injury	32.1
Property damage only	67.9
Total	100.0

[a] Based on HSIS data for Washington (2002–2006)

Table 10-4. Default Distribution by Collision Type for Specific Crash Severity Levels on Rural Two-Lane, Two-Way Roadway Segments

	Percentage of Total Roadway Segment Crashes by Crash Severity Level[a]		
Collision Type	**Total Fatal and Injury**	**Property Damage Only**	**Total (All Severity Levels Combined)**
SINGLE-VEHICLE CRASHES			
Collision with animal	3.8	18.4	12.1
Collision with bicycle	0.4	0.1	0.2
Collision with pedestrian	0.7	0.1	0.3
Overturned	3.7	1.5	2.5
Ran off road	54.5	50.5	52.1
Other single-vehicle crash	0.7	2.9	2.1
Total single-vehicle crashes	63.8	73.5	69.3
MULTIPLE-VEHICLE CRASHES			
Angle collision	10.0	7.2	8.5
Head-on collision	3.4	0.3	1.6
Rear-end collision	16.4	12.2	14.2
Sideswipe collision[b]	3.8	3.8	3.7
Other multiple-vehicle collision	2.6	3.0	2.7
Total multiple-vehicle crashes	36.2	26.5	30.7
Total Crashes	100.0	100.0	100.0

[a] Based on HSIS data for Washington (2002-2006)
[b] Includes approximately 70 percent opposite-direction sideswipe collisions and 30 percent same-direction sideswipe collisions

10.6.2. Safety Performance Functions for Intersections

The predictive model for predicting average crash frequency at particular rural two-lane, two-way road intersections was presented in Equation 10-3. The effect of the major and minor road traffic volumes (AADTs) on crash frequency is incorporated through SPFs, while the effects of geometric design and traffic control features are incorporated through the CMFs. The SPFs for rural two-lane, two-way highway intersections are presented in this section.

Table 10-7. Summary of Crash Modification Factors (CMFs) in Chapter 10 and the Corresponding Safety Performance Functions (SPFs)

Facility Type	CMF	CMF Description	CMF Equations and Tables
Rural Two-Lane Two-Way Roadway Segments	CMF_{1r}	Lane Width	Table 10-8, Figure 10-7, Equation 10-11
	CMF_{2r}	Shoulder Width and Type	Tables 10-9, 10-10, Figure 10-8, Equation 10-12
	CMF_{3r}	Horizontal Curves: Length, Radius, and Presence or Absence of Spiral Transitions	Equation 10-13
	CMF_{4r}	Horizontal Curves: Superelevation	Equations 10-14, 10-15, 10-16
	CMF_{5r}	Grades	Table 10-11
	CMF_{6r}	Driveway Density	Equation 10-17
	CMF_{7r}	Centerline Rumble Strips	See text
	CMF_{8r}	Passing Lanes	See text
	CMF_{9r}	Two-Way Left-Turn Lanes	Equations 10-18, 10-19
	CMF_{10r}	Roadside Design	Equation 10-20
	CMF_{11r}	Lighting	Equations 10-21, Table 10-12
	CMF_{12r}	Automated Speed Enforcement	See text
Three- and four-leg stop control intersections and four-leg signalized intersections	CMF_{1i}	Intersection Skew Angle	Equations 10-22, 10-23
	CMF_{2i}	Intersection Left-Turn Lanes	Table 10-13
	CMF_{3i}	Intersection Right-Turn Lanes	Table 10-14
	CMF_{4i}	Lighting	Equation 10-24, Table 10-15

10.7.1. Crash Modification Factors for Roadway Segments

The CMFs for geometric design and traffic control features of rural two-lane, two-way roadway segments are presented below. These CMFs are applied in Step 10 of the predictive method and used in Equation 10-2 to adjust the SPF for rural two-lane, two-way roadway segments presented in Equation 10-6, to account for differences between the base conditions and the local site conditions.

CMF_{1r}—Lane Width

The CMF for lane width on two-lane highway segments is presented in Table 10-8 and illustrated by the graph in Figure 10-7. This CMF was developed from the work of Zegeer et al. (16) and Griffin and Mak (4). The base value for the lane width CMF is 12 ft. In other words, the roadway segment SPF will predict safety performance of a roadway segment with 12-ft lanes. To predict the safety performance of the actual segment in question (e.g., one with lane widths different than 12 ft), CMFs are used to account for differences between base and actual conditions. Thus, 12-ft lanes are assigned a CMF of 1.00. CMF_{1r} is determined from Table 10-8 based on the applicable lane width and traffic volume range. The relationships shown in Table 10-8 are illustrated in Figure 10-7. Lanes with widths greater than 12 ft are assigned a CMF equal to that for 12-ft lanes.

For lane widths with 0.5-ft increments that are not depicted specifically in Table 10-8 or Figure 10-7, a CMF value can be interpolated using either of these exhibits since there is a linear transition between the various AADT effects.

Where:

CMF_{1r} = crash modification factor for the effect of lane width on total crashes;

CMF_{ra} = crash modification factor for the effect of lane width on related crashes (i.e., single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes), such as the crash modification factor for lane width shown in Table 10-8; and

p_{ra} = proportion of total crashes constituted by related crashes.

The proportion of related crashes, p_{ra}, (i.e., single-vehicle run-off-the-road, and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipes crashes) is estimated as 0.574 (i.e., 57.4 percent) based on the default distribution of crash types presented in Table 10-4. This default crash type distribution, and therefore the value of p_{ra}, may be updated from local data as part of the calibration process.

CMF_{2r}—Shoulder Width and Type

The CMF for shoulders has a CMF for shoulder width (CMF_{wra}) and a CMF for shoulder type (CMF_{tra}). The CMFs for both shoulder width and shoulder type are based on the results of Zegeer et al. (16, 17). The base value of shoulder width and type is a 6-foot paved shoulder, which is assigned a CMF value of 1.00.

CMF_{wra} for shoulder width on two-lane highway segments is determined from Table 10-9 based on the applicable shoulder width and traffic volume range. The relationships shown in Table 10-9 are illustrated in Figure 10-8.

Shoulders over 8-ft wide are assigned a CMF_{wra} equal to that for 8-ft shoulders. The CMFs shown in Table 10-9 and Figure 10-8 apply only to single-vehicle run-off the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes.

Table 10-9. CMF for Shoulder Width on Roadway Segments (CMF_{wra})

	AADT (vehicles per day)		
Shoulder Width	**< 400**	**400 to 2000**	**> 2000**
0 ft	1.10	$1.10 + 2.5 \times 10^{-4}$ (AADT − 400)	1.50
2 ft	1.07	$1.07 + 1.43 \times 10^{-4}$ (AADT − 400)	1.30
4 ft	1.02	$1.02 + 8.125 \times 10^{-5}$ (AADT − 400)	1.15
6 ft	1.00	1.00	1.00
8 ft or more	0.98	$0.98 - 6.875 \times 10^{-5}$ (AADT − 400)	0.87

Note: The collision types related to shoulder width to which this CMF applies include single-vehicle run-off the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes.

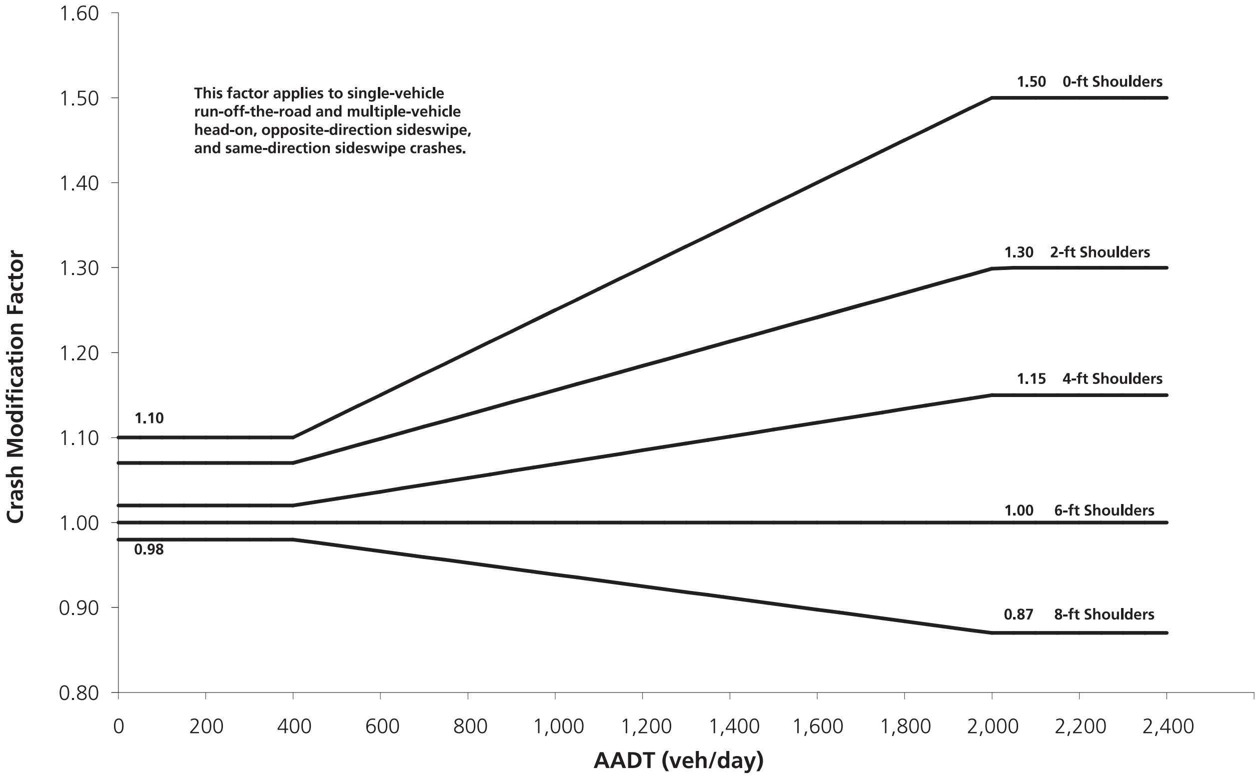


Figure 10-8. Crash Modification Factor for Shoulder Width on Roadway Segments

The base condition for shoulder type is paved. Table 10-10 presents values for CMF_{tra} which adjusts for the safety effects of gravel, turf, and composite shoulders as a function of shoulder width.

Table 10-10. Crash Modification Factors for Shoulder Types and Shoulder Widths on Roadway Segments (CMF_{tra})

	Shoulder Width (ft)						
Shoulder Type	**0**	**1**	**2**	**3**	**4**	**6**	**8**
Paved	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Gravel	1.00	1.00	1.01	1.01	1.01	1.02	1.02
Composite	1.00	1.01	1.02	1.02	1.03	1.04	1.06
Turf	1.00	1.01	1.03	1.04	1.05	1.08	1.11

Note: The values for composite shoulders in this table represent a shoulder for which 50 percent of the shoulder width is paved and 50 percent of the shoulder width is turf.

If the shoulder types and/or widths for the two directions of a roadway segment differ, the CMF are determined separately for the shoulder type and width in each direction of travel and the resulting CMFs are then be averaged.

The CMFs for shoulder width and type shown in Tables 10-9 and 10-10, and Figure 10-8 apply only to the collision types that are most likely to be affected by shoulder width and type: single-vehicle run-off the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes. The CMFs expressed on this basis are, therefore, adjusted to total crashes using Equation 10-12.

Worksheet SP1E—Summary Results or Rural Two-Lane, Two-Way Roadway Segments

Worksheet SP1E presents a summary of the results. Using the roadway segment length, the worksheet presents the crash rate in miles per year (Column 5).

Worksheet SP1E. Summary Results for Rural Two-Lane, Two-Way Roadway Segments

(1)	(2)	(3)	(4)	(5)
Crash Severity Level	**Crash Severity Distribution**	**Predicted Average Crash Frequency (crashes/year)**	**Roadway Segment Length (mi)**	**Crash Rate (crashes/mi/year)**
	(4) from Worksheet SP1C	(8) from Worksheet SP1C		(3)/(4)
Total	1.000	6.084	1.5	4.1
Fatal and injury (FI)	0.321	1.954	1.5	1.3
Property damage only (PDO)	0.679	4.131	1.5	2.8

10.12.2. Sample Problem 2

The Site/Facility

A rural two-lane curved roadway segment.

The Question

What is the predicted average crash frequency of the roadway segment for a particular year?

The Facts

- 0.1-mi length
- Curved roadway segment
- 8,000 veh/day
- 1% grade
- 1,200-ft horizontal curve radius
- No spiral transition
- 0 driveways per mi
- 11-ft lane width
- 2-ft gravel shoulder
- Roadside hazard rating = 5
- 0.1-mi horizontal curve length
- 0.04 superelevation rate

Assumptions

Collision type distributions have been adapted to local experience. The percentage of total crashes representing single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes is 78 percent.

Worksheet SP6A. Predicted and Observed Crashes by Severity and Site Type Using the Project-Level EB Method for Rural Two-Lane, Two-Way Roads and Multilane Highways

(1)	(2)	(3)	(4)	(5)	(6)
	Predicted Average Crash Frequency (crashes/year)			Observed Crashes, $N_{observed}$ (crashes/year)	Overdispersion Parameter, k
Site Type	$N_{predicted\ (total)}$	$N_{predicted\ (FI)}$	$N_{predicted\ (PDO)}$		
ROADWAY SEGMENTS					
Segment 1	6.084	1.954	4.131	—	0.16
Segment 2	0.525	0.169	0.356	—	2.36
INTERSECTIONS					
Intersection 1	2.857	1.186	1.671	—	0.54
Combined (Sum of Column)	9.466	3.309	6.158	15	—

Worksheet SP6A continued

	(7)	(8)	(9)	(10)	(11)	(12)	(13)
	$N_{predicted\ w0}$	$N_{predicted\ w1}$	W_0	N_0	w_1	N_1	$N_{expected/comb}$
Site Type	Equation A-8 (6)*(2)²	Equation A-9 sqrt((6)*(2))	Equation A-10	Equation A-11	Equation A-12	Equation A-13	Equation A-14
ROADWAY SEGMENTS							
Segment 1	5.922	0.987	—	—	—	—	—
Segment 2	0.651	1.113	—	—	—	—	—
INTERSECTIONS							
Intersection 1	4.408	1.242	—	—	—	—	—
Combined (Sum of Column)	10.981	3.342	0.463	12.438	0.739	10.910	11.674

Note: $N_{predicted\ w0}$ = Predicted number of total crashes assuming that crash frequencies are statistically independent

$$N_{\text{predicted } w0} = \sum_{j=1}^{5} k_{rmj} N_{rmj}^2 + \sum_{j=1}^{5} k_{rsj} N_{rsj}^2 + \sum_{j=1}^{5} k_{rdj} N_{rdj}^2 + \sum_{j=1}^{4} k_{imj} N_{imj}^2 + \sum_{j=1}^{4} k_{isj} N_{isj}^2 \quad \text{(A-8)}$$

$N_{predicted\ w1}$ = Predicted number of total crashes assuming that crash frequencies are perfectly correlated

$$N_{\text{predicted } w1} = \sum_{i=1}^{5} \sqrt{k_{rmj} N_{rmj}} + \sum_{i=1}^{5} \sqrt{k_{rsj} N_{rsj}} + \sum_{i=1}^{5} \sqrt{k_{rdj} N_{rdj}} + \sum_{i=1}^{4} \sqrt{k_{imj} N_{imj}} + \sum_{i=1}^{4} \sqrt{k_{isj} N_{isj}} \quad \text{(A-9)}$$

Column 9—w_0

The weight placed on predicted crash frequency under the assumption that crashes frequencies for different roadway elements are statistically independent, w_0, is calculated using Equation A-10 as follows:

$$w_0 = \frac{1}{1 + \frac{N_{\text{predicted } w0}}{N_{\text{predicted (total)}}}} \quad \text{(A-10)}$$

$$= \frac{1}{1 + \frac{10.981}{9.466}}$$

$$= 0.463$$

Column 10—N_0

The expected crash frequency based on the assumption that different roadway elements are statistically independent, N_0, is calculated using Equation A-11 as follows:

$$N_0 = w_0 \times N_{predicted(total)} + (1 - w_0) \times N_{observed(total)} \tag{A-11}$$

$$= 0.463 \times 9.466 + (1 - 0.463) \times 15 = 12.438$$

Column 11—w_1

The weight placed on predicted crash frequency under the assumption that crashes frequencies for different roadway elements are perfectly correlated, w_1, is calculated using Equation A-12 as follows:

$$w_1 = \frac{1}{1 + \frac{N_{predicted\ w1}}{N_{predicted\ (total)}}} \tag{A-12}$$

$$= \frac{1}{1 + \frac{3.342}{9.466}}$$

$$= 0.739$$

Column 12—N_1

The expected crash frequency based on the assumption that different roadway elements are perfectly correlated, N_1, is calculated using Equation A-13 as follows:

$$N_1 = w_1 \times N_{predicted(total)} + (1 - w_1) \times N_{observed(total)} \tag{A-13}$$

$$= 0.739 \times 9.466 + (1 - 0.739) \times 15 = 10.910$$

Column 13—$N_{expected/comb}$

The expected average crash frequency based of combined sites, $N_{expected/comb}$, is calculated using Equation A-14 as follows:

$$N_{expected/comb} = \frac{N_0 + N_1}{2} \tag{A-14}$$

$$= \frac{12.438 + 10.910}{2}$$

$$= 11.674$$

Worksheet SP6B—Project-Level EB Method Summary Results for Rural Two-Lane, Two-Way Roads and Multilane Highways

Worksheet SP6B presents a summary of the results. The expected average crash frequency by severity level is calculated by applying the proportion of predicted average crash frequency by severity level to the total expected average crash frequency (Column 3).

An overview of the use of calibration factors is provided in Section C.6.5. Detailed guidance for the development of calibration factors is included in Part C, Appendix A.1.1.

Steps 9, 10, and 11 together implement the predictive models in Equations 11-2, 11-3, and 11-4 to determine predicted average crash frequency.

Step 12—If there is another year to be evaluated in the study period for the selected site, return to Step 8. Otherwise, proceed to Step 13.
This step creates a loop through Steps 8 to 12 that is repeated for each year of the evaluation period for the selected site.

Step 13—Apply site-specific EB Method (if applicable).
Whether the site-specific EB Method is applicable is determined in Step 3. The site-specific EB Method combines the Chapter 11 predictive model estimate of predicted average crash frequency, $N_{predicted}$, with the observed crash frequency of the specific site, $N_{observed}$. This provides a more statistically reliable estimate of the expected average crash frequency of the selected site.

In order to apply the site-specific EB Method, overdispersion parameter, *k*, for the SPF is used. This is in addition to the material in Part C, Appendix A.2.4. The overdispersion parameter provides an indication of the statistical reliability of the SPF. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. This parameter is used in the site-specific EB Method to provide a weighting to $N_{predicted}$ and $N_{observed}$. Overdispersion parameters are provided for each SPF in Section 11.6.

Apply the site-specific EB Method to a future time period, if appropriate.

The estimated expected average crash frequency obtained above applies to the time period in the past for which the observed crash data were obtained. Part C, Appendix A.2.6 provides a method to convert the estimate of expected average crash frequency for a past time period to a future time period.

Step 14—If there is another site to be evaluated, return to Step 7, otherwise, proceed to Step 15.
This step creates a loop through Steps 7 to 13 that is repeated for each roadway segment or intersection within the facility.

Step 15—Apply the project level EB Method (if the site specific EB Method is not applicable).
This step is only applicable to existing conditions when observed crash data are available but cannot be accurately assigned to specific sites (e.g., the crash report may identify crashes as occurring between two intersections, but is not accurate to determine a precise location on the segment). Detailed description of the project level EB Method is provided in Part C, Appendix A.2.5.

Step 16—Sum all sites and years in the study to estimate total crash frequency.
The total estimated number of crashes within the network or facility limits during a study period of *n* years is calculated using Equation 11-5:

$$N_{\text{total}} = \sum_{\substack{\text{all}\\\text{roadway}\\\text{segments}}} N_{rs} + \sum_{\substack{\text{all}\\\text{intersections}}} N_{int} \tag{11-5}$$

Where:

N_{total} = total expected number of crashes within the limits of a rural multilane highway for the period of interest. Or, the sum of the expected average crash frequency for each year for each site within the defined roadway limits within the study period;

N_{rs} = expected average crash frequency for a roadway segment using the predictive method for one specific year; and

N_{int} = expected average crash frequency for an intersection using the predictive method for one specific year.

Equation 11-5 represents the total expected number of crashes estimated to occur during the study period. Equation 11-6 is used to estimate the total expected average crash frequency within the network or facility limits during the study period.

$$N_{\text{total average}} = \frac{N_{\text{total}}}{n} \tag{11-6}$$

Where:

$N_{\text{total average}}$ = total expected average crash frequency estimated to occur within the defined network or facility limits during the study period; and

n = number of years in the study period.

Step 17—Determine if there is an alternative design, treatment, or forecast AADT to be evaluated.
Steps 3 through 16 of the predictive method are repeated as appropriate for the same roadway limits but for alternative conditions, treatments, periods of interest, or forecast AADTs.

Step 18—Evaluate and compare results.
The predictive method is used to provide a statistically reliable estimate of the expected average crash frequency within defined network or facility limits over a given period of time, for given geometric design and traffic control features, and known or estimated AADT. In addition to estimating total crashes, the estimate can be made for different crash severity types and different collision types. Default distributions of crash severity and collision type are provided with each SPF in Section 11.6. These default distributions can benefit from being updated based on local data as part of the calibration process presented in Part C, Appendix A.1.

11.5. ROADWAY SEGMENTS AND INTERSECTIONS

Section 11.4 provides an explanation of the predictive method. Sections 11.5 through 11.8 provide the specific detail necessary to apply the predictive method steps on rural multilane roads. Detail regarding the procedure for determining a calibration factor to apply in Step 11 is provided in Part C, Appendix A.1. Detail regarding the EB Method, which is applied in Steps 6, 13, and 15, is provided in Part C, Appendix A.2.

In Step 5 of the predictive method, the roadway within the defined roadway limits is divided into individual sites, which are homogenous roadway segments and intersections. A facility consists of a contiguous set of individual intersections and roadway segments, referred to as "sites." A roadway network consists of a number of contiguous facilities. Predictive models have been developed to estimate crash frequencies separately for roadway segments and intersections. The definitions of roadway segments and intersections presented below are the same as those for used in the FHWA *Interactive Highway Safety Design Model* (IHSDM) (2).

Roadway segments begin at the center of an intersection and end at either the center of the next intersection or where there is a change from one homogeneous roadway segment to another homogenous segment. The roadway segment model estimates the frequency of roadway-segment-related crashes which occur in Region B in Figure 11-2. When a roadway segment begins or ends at an intersection, the length of the roadway segment is measured from the center of the intersection.

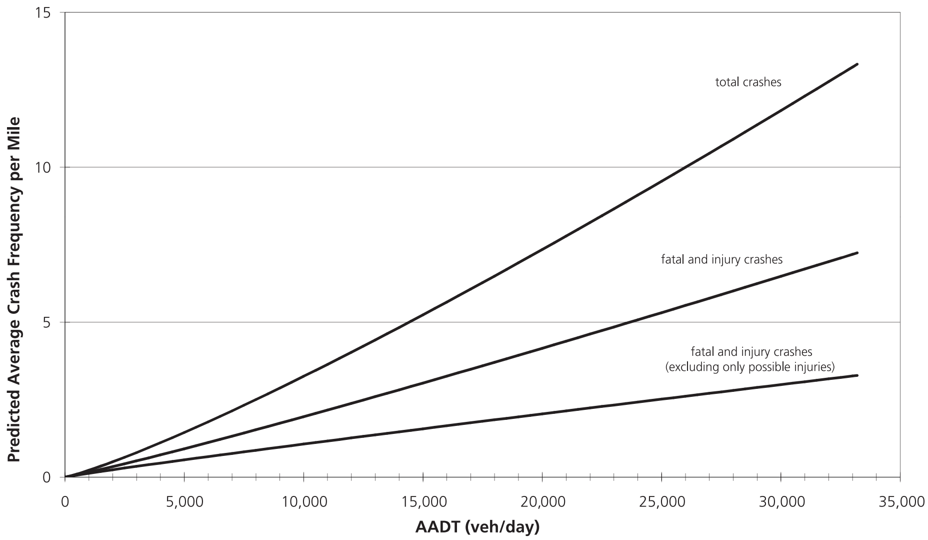


Figure 11-3. Graphical Form of the SPF for Undivided Roadway Segments (from Equation 11-7 and Table 11-3)

The default proportions in Table 11-4 are used to break down the crash frequencies from Equation 11-7 into specific collision types. To do so, the user multiplies the crash frequency for a specific severity level from Equation 11-7 by the appropriate collision type proportion for that severity level from Table 11-4 to estimate the number of crashes for that collision type. Table 11-4 is intended to separate the predicted frequencies for total crashes (all severity levels combined), fatal-and-injury crashes, and fatal-and-injury crashes (with possible injuries excluded) into components by collision type. Table 11-4 cannot be used to separate predicted total crash frequencies into components by severity level. Ratios for PDO crashes are provided for application where the user has access to predictive models for that severity level. The default collision type proportions shown in Table 11-4 may be updated with local data.

There are a variety of factors that may affect the distribution of crashes among crash types and severity levels. To account for potential differences in these factors between jurisdictions, it is recommended that the values in Table 11-4 be updated with local data. The values for total, fatal-and-injury, and fatal-and-injury (with possible injuries excluded) crashes in this exhibit are used in the worksheets described in Appendix 11A.

The SPF for expected average crash frequency for divided roadway segments on rural multilane highways is shown in Equation 11-9 and presented graphically in Figure 11-4:

$$N_{spf\,rd} = e^{(a + b \times \ln(AADT) + \ln(L))} \tag{11-9}$$

Where:

$N_{spf\,rd}$ = base total number of roadway segment crashes per year;

$AADT$ = annual average daily traffic (vehicles/day) on roadway segment;

L = length of roadway segment (miles); and

a, b = regression coefficients.

Guidance on the estimation of traffic volumes for roadway segments for use in the SPFs is presented in Step 3 of the predictive method described in Section 11.4. The SPFs for divided roadway segments on rural multilane highways are applicable to the AADT range from zero to 89,300 vehicles per day. Application to sites with AADTs substantially outside this range may not provide reliable results.

The value of the overdispersion parameter is determined as a function of segment length as:

$$k = \frac{1}{e^{(c+\ln(L))}} \tag{11-10}$$

Where:

k = overdispersion parameter associated with the roadway segment;

L = length of roadway segment (mi); and

c = a regression coefficient used to determine the overdispersion parameter.

Table 11-5 presents the values for the coefficients used in applying Equations 11-9 and 11-10.

Table 11-5. SPF Coefficients for Total and Fatal-and-Injury Crashes on Divided Roadway Segments (for use in Equations 11-9 and 11-10)

Severity Level	a	b	c
4-lane total	–9.025	1.049	1.549
4-lane fatal and injury	–8.837	0.958	1.687
4-lane fatal and injury[a]	–8.505	0.874	1.740

[a] Using the KABCO scale, these include only KAB crashes. Crashes with severity level C (possible injury) are not included.

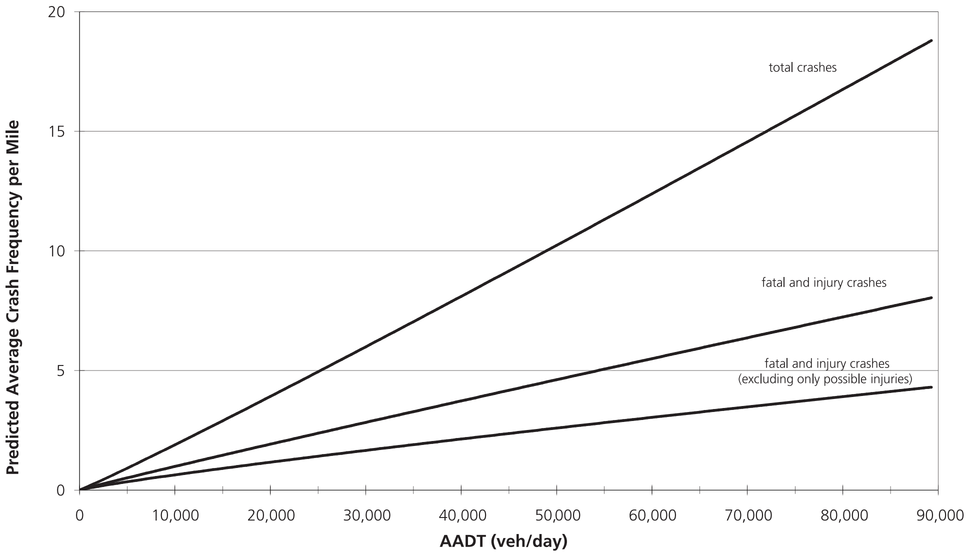


Figure 11-4. Graphical Form of SPF for Rural Multilane Divided Roadway Segments (from Equation 11-9 and Table 11-5)

The default proportions in Table 11-6 are used to break down the crash frequencies from Equation 11-9 into specific collision types. To do so, the user multiplies the crash frequency for a specific severity level from Equation 11-9 by the appropriate collision type proportion for that severity level from Table 11-6 to estimate the number of crashes for that collision type. Table 11-6 is intended to separate the predicted frequencies for total crashes (all severity levels combined), fatal-and-injury crashes, and fatal-and-injury crashes (with possible injuries excluded) into components by collision type. Table 11-6 cannot be used to separate predicted total crash frequencies into components by severity level. Ratios for property-damage-only (PDO) crashes are provided for application where the user has access to predictive models for that severity level. The default collision type proportions shown in Table 11-6 may be updated with local data.

Table 11-17. CMF for Right Shoulder Width on Divided Roadway Segments (CMF_{2rd})

Average Shoulder Width (ft)				
0	**2**	**4**	**6**	**8 or more**
1.18	1.13	1.09	1.04	1.00

Note: This CMF applies to paved shoulders only.

CMF_{3rd}—Median Width

A CMF for median widths on divided roadway segments of rural multilane highways is presented in Table 11-18 based on the work of Harkey et al. (3). The median width of a divided highway is measured between the inside edges of the through travel lanes in the opposing direction of travel; thus, inside shoulder and turning lanes are included in the median width. The base condition for this CMF is a median width of 30 ft. The CMF applies to total crashes, but represents the effect of median width in reducing cross-median collisions; the CMF assumes that nonintersection collision types other than cross-median collisions are not affected by median width. The CMF in Table 11-18 has been adapted from the CMF in Table 13-13 based on the estimate by Harkey et al. (3) that cross-median collisions represent 12.2 percent of crashes on multilane divided highways.

This CMF applies only to traversable medians without traffic barriers. The effect of traffic barriers on safety would be expected to be a function of the barrier type and offset, rather than the median width; however, the effects of these factors on safety have not been quantified. Until better information is available, a CMF value of 1.00 is used for medians with traffic barriers.

Table 11-18. CMFs for Median Width on Divided Roadway Segments without a Median Barrier (CMF_{3rd})

Median Width (ft)	CMF
10	1.04
20	1.02
30	1.00
40	0.99
50	0.97
60	0.96
70	0.96
80	0.95
90	0.94
100	0.94

Note: This CMF applies only to medians without traffic barriers.

CMF_{4rd}—Lighting

The SPF base condition for lighting is the absence of roadway segment lighting. The CMF for lighted roadway segments is determined, based on the work of Elvik and Vaa (1), as:

$$CMF_{4rd} = 1 - [(1 - 0.72 \times p_{inr} - 0.83 \times p_{pnr}) \times p_{nr}] \quad (11\text{-}17)$$

Where:

CMF_{4rd} = crash modification factor for the effect of lighting on total crashes;

p_{inr} = proportion of total nighttime crashes for unlighted roadway segments that involve a fatality or injury;

p_{pnr} = proportion of total nighttime crashes for unlighted roadway segments that involve property damage only; and

p_{nr} = proportion of total crashes for unlighted roadway segments that occur at night.

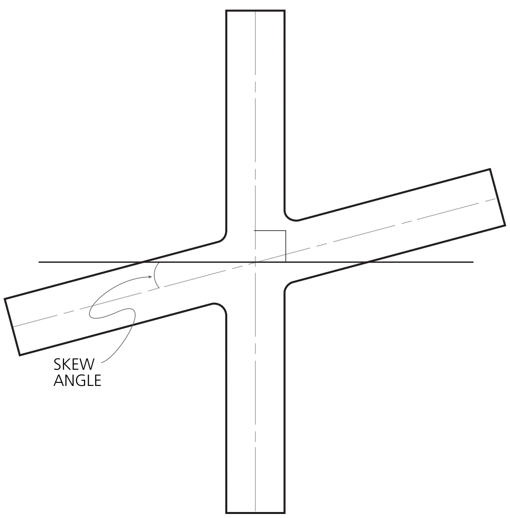
Table 11-21. CMFs for Four-Leg Intersection with Minor-Road Stop Control (4ST)

CMFs	Total	Fatal and Injury
Intersection Angle	Equation 11-20	Equation 11-21
Left-Turn Lane on Major Road	Table 11-22	Table 11-22
Right-Turn Lane on Major Road	Table 11-23	Table 11-23
Lighting	Equation 11-22	Equation 11-22

CMF_{1i}—Intersection Skew Angle

The SPF base condition for intersection skew angle is 0 degrees of skew (i.e., an intersection angle of 90 degrees). Reducing the skew angle of three- or four-leg stop-controlled intersections on rural multilane highways reduces total intersection crashes, as shown below. The skew angle is the deviation from an intersection angle of 90 degrees. Skew carries a positive or negative sign that indicates whether the minor road intersects the major road at an acute or obtuse angle, respectively.

Illustration of Intersection Skew Angle



Three-Leg Intersections with Stop-Control on the Minor Approach

The CMF for total crashes for intersection skew angle at three-leg intersections with stop-control on the minor approach is:

$$CMF_{1i} = \frac{0.016 \times skew}{(0.98 + 0.016 \times skew} + 1 \quad (11\text{-}18)$$

and the CMF for fatal-and-injury crashes is:

$$CMF_{1i} = \frac{0.017 \times skew}{(0.52 + 0.017 \times skew} + 1 \quad (11\text{-}19)$$

Where:

CMF_{1i} = crash modification factor for the effect of intersection skew on total crashes; and

skew = intersection skew angle (in degrees); the absolute value of the difference between 90 degrees and the actual intersection angle.

Four-Leg Intersections with Stop-Control on the Minor Approaches

The CMF for total crashes for intersection angle at four-leg intersection with stop-control on the minor approaches is:

$$CMF_{1i} = \frac{0.053 \times skew}{(1.43 + 0.053 \times skew)} + 1.0 \tag{11-20}$$

The CMF for fatal-and-injury crashes is:

$$CMF_{1i} = \frac{0.048 \times skew}{(0.72 + 0.048 \times skew)} + 1.0 \tag{11-21}$$

CMF_{2i}—Intersection Left-Turn Lanes

The SPF base condition for intersection left-turn lanes is the absence of left-turn lanes on all of the intersection approaches. The CMFs for presence of left-turn lanes are presented in Table 11-22 for total crashes and injury crashes. These CMFs apply only on uncontrolled major-road approaches to stop-controlled intersections. The CMFs for installation of left-turn lanes on multiple approaches to an intersection are equal to the corresponding CMF for installation of a left-turn lane on one approach raised to a power equal to the number of approaches with left-turn lanes (i.e., the CMFs are multiplicative, and Equation 3-7 can be used). There is no indication of any effect of providing a left-turn lane on an approach controlled by a stop sign, so the presence of a left-turn lane on a stop-controlled approach is not considered in applying Table 11-22. The CMFs for installation of left-turn lanes are based on research by Harwood et al. (4) and are consistent with the CMFs presented in Chapter 14, Intersections. A CMF of 1.00 is used when no left-turn lanes are present.

Table 11-22. Crash Modification Factors (CMF_{2i}) for Installation of Left-Turn Lanes on Intersection Approaches

		Number of Non-Stop-Controlled Approaches with Left-Turn Lanes[a]	
Intersection Type	**Crash Severity Level**	**One Approach**	**Two Approaches**
Three-leg minor-road stop control[b]	Total	0.56	—
	Fatal and Injury	0.45	—
Four-leg minor-road stop control[b]	Total	0.72	0.52
	Fatal and Injury	0.65	0.42

[a] Stop-controlled approaches are not considered in determining the number of approaches with left-turn lanes
[b] Stop signs present on minor-road approaches only.

CMF_{3i}—Intersection Right-Turn Lanes

The SPF base condition for intersection right-turn lanes is the absence of right-turn lanes on the intersection approaches. The CMFs for the presence of right-turn lanes are based on research by Harwood et al. (4) and are consistent with the CMFs in Chapter 14. These CMFs apply to installation of right-turn lanes on any approach to a signalized intersection, but only on uncontrolled major-road approaches to stop-controlled intersections. The CMFs for installation of right-turn lanes on multiple approaches to an intersection are equal to the corresponding CMF for installation of a right-turn lane on one approach raised to a power equal to the number of approaches with right-turn lanes (i.e., the CMFs are multiplicative, and Equation 3-7 can be used). There is no indication of any safety effect for providing a right-turn lane on an approach controlled by a stop sign, so the presence of a right-turn lane on a stop-controlled approach is not considered in applying Table 11-23. The CMFs for presence of right-turn lanes are presented in Table 11-23 for total crashes and injury crashes. A CMF value of 1.00 is used when no right-turn lanes are present. This CMF applies only to right-turn lanes that are identified by marking or signing. The CMF is not applicable to long tapers, flares, or paved shoulders that may be used informally by right-turn traffic.

Table 11-23. Crash Modification Factors (CMF_{3i}) for Installation of Right-Turn Lanes on Intersections Approaches

Intersection Type	Crash Severity Level	Number of Non-Stop-Controlled Approaches with Right-Turn Lanes[a]	
		One Approach	Two Approaches
Three-leg minor-road stop control[b]	Total	0.86	—
	Fatal and Injury	0.77	—
Four-leg minor-road stop control[b]	Total	0.86	0.74
	Fatal and Injury	0.77	0.59

[a] Stop-controlled approaches are not considered in determining the number of approaches with right-turn lanes.
[b] Stop signs present on minor-road approaches only.

CMF_{4i}—Lighting

The SPF base condition for lighting is the absence of intersection lighting. The CMF for lighted intersections is adapted from the work of Elvik and Vaa (1), as:

$$CMF_{4i} = 1.0 - 0.38 \times p_{ni} \quad (11\text{-}22)$$

Where:

CMF_{4i} = crash modification factor for the effect of lighting on total crashes; and

p_{ni} = proportion of total crashes for unlighted intersections that occur at night.

This CMF applies to total intersections crashes. Table 11-24 presents default values for the nighttime crash proportion, p_{ni}. HSM users are encouraged to replace the estimates in Table 11-24 with locally derived values.

Table 11-24. Default Nighttime Crash Proportions for Unlighted Intersections

Intersection Type	Proportion of Crashes that Occur at Night, p_{ni}
3ST	0.276
4ST	0.273

11.8. CALIBRATION TO LOCAL CONDITIONS

In Step 10 of the predictive method, presented in Section 11.4, the predictive model is calibrated to local state or geographic conditions. Crash frequencies, even for nominally similar roadway segments or intersections, can vary widely from one jurisdiction to another. Geographic regions differ markedly in climate, animal population, driver populations, crash-reporting threshold, and crash-reporting practices. These variations may result in some jurisdictions experiencing a different number of traffic crashes on rural multilane highways than others. Calibration factors are included in the methodology to allow highway agencies to adjust the SPFs to match actual local conditions.

The calibration factors for roadway segments and intersections (defined below as C_r and C_i, respectively) will have values greater than 1.0 for roadways that, on average, experience more crashes than the roadways used in the development of the SPFs. The calibration factors for roadways that experience fewer crashes on average than the roadways used in the development of the SPFs will have values less than 1.0. The calibration procedures are presented in Part C, Appendix A.

Step 10—Multiply the result obtained in Step 9 by the appropriate CMFs to adjust base conditions to site specific geometric conditions and traffic control features

Each CMF used in the calculation of the predicted average crash frequency of the intersection is calculated below:

Intersection Skew Angle (CMF_{1i})

CMF_{1i} can be calculated from Equation 11-18 as follows:

$$CMF_{1i} = \frac{0.016 \times skew}{(0.98 + 0.016 \times skew} + 1$$

The intersection skew angle for Sample Problem 3 is 30 degrees.

$$CMF_{1i} = \frac{0.016 \times 30}{(0.98 + 0.016 \times 30)} + 1 = 1.08$$

Intersection Left-Turn Lanes (CMF_{2i})

From Table 11-22, for a left-turn lane on one non-stop-controlled approach at a three-leg stop-controlled intersection, $CMF_{2i} = 0.56$.

Intersection Right-Turn Lanes (CMF_{3i})

Since no right-turn lanes are present, $CMF_{3i} = 1.00$ (i.e., the base condition for CMF_{3i} is the absence of right-turn lanes on the intersection approaches).

Lighting (CMF_{4i})

CMF_{4i} can be calculated from Equation 11-22 as follows:

$$CMF_{4i} = 1.0 - 0.38 \times p_{ni}$$

From Table 11-24, for intersection lighting at a three-leg stop-controlled intersection, $p_{ni} = 0.276$.

$$CMF_{4i} = 1.0 - 0.38 \times 0.276 = 0.90$$

The combined CMF value for Sample Problem 3 is calculated below.

$$CMF_{comb} = 1.33 \times 0.56 \times 0.90 = 0.67$$

Step 11—Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed that a calibration factor, C_i, of 1.50 has been determined for local conditions. See Part C, Appendix A.1 for further discussion on calibration of the predictive models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated using Equation 11-4 based on the results obtained in Steps 9 through 11 as follows:

$$N_{predicted\ int} = N_{spf\ int} \times C_i \times (CMF_{1i} \times CMF_{2i} \times \ldots \times CMF_{4i})$$

$$= 0.928 \times 1.50 \times (0.67) = 0.933 \text{ crashes/year}$$

Worksheet SP3B. Crash Modification Factors for Rural Multilane Highway Intersections

(1)	(2)	(3)	(4)	(5)	(6)
	CMF for Intersection Skew Angle	CMF for Left-Turn Lanes	CMF for Right-Turn Lanes	CMF for Lighting	Combined CMF
	CMF_{1i}	CMF_{2i}	CMF_{3i}	CMF_{4i}	CMF_{comb}
Crash Severity Level	from Equations 11-18 or 11-20 and 11-19 or 11-21	from Table 11-22	from Table 11-23	from Equation 11-22	(1)*(2)*(3)*(4)
Total	1.33	0.56	1.00	0.90	0.67
Fatal and injury (FI)	1.50	0.45	1.00	0.90	0.61

Worksheet SP3C—Intersection Crashes for Rural Multilane Highway Intersections

The SPF for the intersection in Sample Problem 3 is calculated using the coefficients shown in Table 11-7 (Column 2), which are entered into Equation 11-11 (Column 3). The overdispersion parameter associated with the SPF is also found in Table 11-7 and entered into Column 4; however, the overdispersion parameter is not needed for Sample Problem 3 (as the EB Method is not utilized). Column 5 represents the combined CMF (from Column 6 in Worksheet SP3B), and Column 6 represents the calibration factor. Column 7 calculates the predicted average crash frequency using the values in Column 3, the combined CMF in Column 5, and the calibration factor in Column 6.

Worksheet SP3C. Intersection Crashes for Rural Multilane Highway Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)
	SPF Coefficients			$N_{spf\ int}$	Overdispersion Parameter, k	Combined CMFs		Predicted Average Crash Frequency, $N_{predicted\ int}$
	from Tables 11-7 or 11-8							
Crash Severity Level	a	b	c	from Equation 11-11 or 11-12	from Tables 11-7 or 11-8	from (6) of Worksheet SP3B	Calibration Factor, C_i	(3)*(5)*(6)
Total	–12.526	1.204	0.236	0.928	0.460	0.67	1.50	0.933
Fatal and injury (FI)	–12.664	1.107	0.272	0.433	0.569	0.61	1.50	0.396
Fatal and injury[a] (FI[a])	–11.989	1.013	0.228	0.270	0.566	0.61	1.50	0.247
Property damage only (PDO)	—	—	—	—	—	—	—	$(7)_{total}–(7)_{FI}$ = 0.537

[a] Using the KABCO scale, these include only KAB crashes. Crashes with severity level C (possible injury) are not included.

Worksheet SP3D—Crashes by Severity Level and Collision Type for Rural Multilane Highway Intersections

Worksheet SP3D presents the default proportions for collision type (from Table 11-9) by crash severity level as follows:

- Total crashes (Column 2)
- Fatal-and-injury crashes (Column 4)
- Fatal-and-injury crashes, not including "possible-injury" crashes (i.e., on a KABCO injury scale, only KAB crashes) (Column 6)
- Property-damage-only crashes (Column 8)

Using the default proportions, the predicted average crash frequency by collision type in Columns 3 (Total), 5 (Fatal and Injury, FI), 7 (Fatal and Injury, not including "possible injury"), and 9 (Property Damage Only, PDO).

These proportions may be used to separate the predicted average crash frequency (from Column 7, Worksheet SP3C) by crash severity and collision type.

Worksheet SP3D. Crashes by Severity Level and Collision Type for Rural Multilane Highway Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Collision Type	**Proportion of Collision Type** $_{(total)}$	$N_{predicted\ int\ (total)}$ **(crashes/year)**	**Proportion of Collision Type** $_{(FI)}$	$N_{predicted\ int\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type** $_{(FI^a)}$	$N_{predicted\ int\ (FI^a)}$ **(crashes/year)**	**Proportion of Collision Type (PDO)**	$N_{predicted\ int\ (PDO)}$ **(crashes/year)**
	from Table 11-9	$(7)_{total}$ **from Worksheet SP3C**	**from Table 11-9**	$(7)_{FI}$ **from Worksheet SP3C**	**from Table 11-9**	$(7)_{FI^a}$ **from Worksheet SP3C**	**from Table 11-9**	$(7)_{PDO}$ **from Worksheet SP3C**
Total	1.000	0.933	1.000	0.396	1.000	0.247	1.000	0.537
		$(2)*(3)_{total}$		$(4)*(5)_{FI}$		$(6)*(7)_{FI^a}$		$(8)*(9)_{PDO}$
Head-on collision	0.029	0.027	0.043	0.017	0.052	0.013	0.020	0.011
Sideswipe collision	0.133	0.124	0.058	0.023	0.057	0.014	0.179	0.096
Rear-end collision	0.289	0.270	0.247	0.098	0.142	0.035	0.315	0.169
Angle collision	0.263	0.245	0.369	0.146	0.381	0.094	0.198	0.106
Single-vehicle collision	0.234	0.218	0.219	0.087	0.284	0.070	0.244	0.131
Other collision	0.052	0.049	0.064	0.025	0.084	0.021	0.044	0.024

[a] Using the KABCO scale, these include only KAB crashes. Crashes with severity level C (possible injury) are not included.

Worksheet SP3E—Summary Results for Rural Multilane Highway Intersections

Worksheet SP3E presents a summary of the results.

Worksheet SP3E. Summary Results for Rural Multilane Highway Intersections

(1)	(2)
	Predicted Average Crash Frequency (crashes/year)
Crash Severity Level	**(7) from Worksheet SP3C**
Total	0.933
Fatal and injury (FI)	0.396
Fatal and injury[a] (FI[a])	0.247
Property damage only (PDO)	0.537

[a] Using the KABCO scale, these include only KAB crashes. Crashes with severity level C (possible injury) are not included.

Worksheet SP4A. Predicted and Observed Crashes by Severity and Site Type Using the Site-Specific EB Method for Rural Two-Lane, Two-Way Roads and Multilane Highways

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Predicted Average Crash Frequency (crashes/year)			Observed Crashes, $N_{observed}$ (crashes/year)	Overdispersion Parameter, *k*	Weighted Adjustment, *w*	Expected Average Crash Frequency, $N_{expected}$
Site Type	$N_{predicted\ (total)}$	$N_{predicted\ (FI)}$	$N_{predicted\ (PDO)}$			Equation A-5	Equation A-4
Roadway Segments							
Segment 1	3.306	1.726	1.580	4	0.142	0.681	3.527
Segment 2	0.289	0.177	0.112	2	1.873	0.649	0.890
Intersections							
Intersection 1	0.933	0.396	0.537	3	0.460	0.700	1.554
Combined (Sum of Column)	4.528	2.299	2.229	9	—	—	5.971

Column 7—Weighted Adjustment

The weighted adjustment, *w*, to be placed on the predictive model estimate is calculated using Equation A-5 as follows:

$$w = \frac{1}{1 + k \times \left(\sum_{\text{all study years}} N_{\text{predicted}} \right)}$$

Segment 1

$$w = \frac{1}{1 + 0.142 \times (3.306)} = 0.681$$

Segment 2

$$w = \frac{1}{1 + 1.873 \times (0.289)} = 0.649$$

Intersection 1

$$w = \frac{1}{1 + 0.460 \times (0.933)} = 0.700$$

Column 8—Expected Average Crash Frequency

The estimate of expected average crash frequency, $N_{expected}$, is calculated using Equation A-4 as follows:

$$N_{expected} = w \times N_{predicted} + (1 - w) \times N_{observed}$$

Segment 1: $N_{expected} = 0.681 \times 3.306 + (1 - 0.681) \times 4 = 3.527$

Segment 2: $N_{expected} = 0.649 \times 0.289 + (1 - 0.649) \times 2 = 0.890$

Intersection 1: $N_{expected} = 0.700 \times 0.933 + (1 - 0.700) \times 3 = 1.554$

Worksheet SP4B—Site-Specific EB Method Summary Results for Rural Two-Lane, Two-Way Roads and Multilane Highways

Worksheet SP4B presents a summary of the results. The expected average crash frequency by severity level is calculated by applying the proportion of predicted average crash frequency by severity level to the total expected average crash frequency (Column 3).

Worksheet SP4B. Site-Specific EB Method Summary Results for Rural Two-Lane, Two-Way Roads and Multilane Highways

(1)	(2)	(3)
Crash Severity Level	$N_{predicted}$	$N_{expected}$
Total	$(2)_{comb}$ from Worksheet SP4A	$(8)_{comb}$ from Worksheet SP4A
	4.528	6.0
Fatal and injury (FI)	$(3)_{comb}$ from Worksheet SP4A	$(3)_{total}*(2)_{FI}/(2)_{total}$
	2.299	3.0
Property damage only (PDO)	$(4)_{comb}$ from Worksheet SP4A	$(3)_{total}*(2)_{PDO}/(2)_{total}$
	2.229	3.0

11.12.5. Sample Problem 5

The Project

A project of interest consists of three sites: a rural four-lane divided highway segment, a rural four-lane undivided highway segment, and a three-leg intersection with minor-road stop control. (This project is a compilation of roadway segments and intersections from Sample Problems 1, 2, and 3.)

The Question

What is the expected average crash frequency of the project for a particular year incorporating both the predicted crash frequencies from Sample Problems 1, 2, and 3 and the observed crash frequencies using the **project-level EB Method?**

The Facts

- 2 roadway segments (4D segment, 4U segment)
- 1 intersection (3ST intersection)
- 9 observed crashes (but no information is available to attribute specific crashes to specific sites within the project)

Worksheet SP5A. Predicted and Observed Crashes by Severity and Site Type Using the Project-Level EB Method for Rural Two-Lane, Two-Way Roads and Multilane Highways

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Predicted Average Crash Frequency (crashes/year)			Observed Crashes, $N_{observed}$ (crashes/year)	Overdispersion Parameter, k	N_{w0}
Site Type	$N_{predicted\ (total)}$	$N_{predicted\ (FI)}$	$N_{predicted\ (PDO)}$			Equation A-8 (6)* (2)^2
Roadway Segments						
Segment 1	3.306	1.726	1.580	4	0.142	1.552
Segment 2	0.289	0.177	0.112	2	1.873	0.156
Intersections						
Intersection 1	0.933	0.396	0.537	3	0.460	0.400
Combined (sum of column)	4.528	2.299	2.229	9	—	2.109

Note: $N_{predicted\ w0}$ = Predicted number of total crashes assuming that crash frequencies are statistically independent

Worksheet SP5A. Continued

(1)	(8)	(9)	(10)	(11)	(12)	(13)
Site Type	N_{w1}	w_0	N_0	w_1	N_1	
Equation A-9 sqrt((6)*(2))	Equation A-10	Equation A-11	Equation A-12	Equation A-13	Equation A-14	$N_{expected/comb}$
Roadway Segments						
Segment 1	0.685	—	—	—	—	—
Segment 2	0.736	—	—	—	—	—
Intersections						
Intersection 1	0.655	—	—	—	—	—
Combined (Sum of Column)	2.076	0.682	5.95	0.686	5.932	5.941

Note: $N_{predicted\ w0}$ = Predicted number of total crashes assuming that crash frequencies are statistically independent

$$N_{\text{predicted } w0} = \sum_{j=1}^{5} k_{rmj} N_{rmj}^2 + \sum_{j=1}^{5} k_{rsj} N_{rsj}^2 + \sum_{j=1}^{5} k_{rdj} N_{rdj}^2 + \sum_{j=1}^{4} k_{imj} N_{imj}^2 + \sum_{j=1}^{4} k_{isj} N_{isj}^2 \qquad \text{(A-8)}$$

$N_{predicted\ w1}$ = Predicted number of total crashes assuming that crash frequencies are perfectly correlated

$$N_{\text{predicted } w1} = \sum_{j=1}^{5} \sqrt{k_{rmj} N_{rmj}} + \sum_{j=1}^{5} \sqrt{k_{rsj} N_{rsj}} + \sum_{j=1}^{5} \sqrt{k_{rdj} N_{rdj}} + \sum_{j=1}^{4} \sqrt{k_{imj} N_{imj}} + \sum_{j=1}^{4} \sqrt{k_{isj} N_{isj}} \qquad \text{(A-9)}$$

Column 9—w_0

The weight placed on predicted crash frequency under the assumption that crashes frequencies for different roadway elements are statistically independent, w_0, is calculated using Equation A-10 as follows:

$$w_0 = \frac{1}{1 + \frac{N_{\text{predicted } w0}}{N_{\text{predicted (total)}}}}$$

$$= \frac{1}{1 + \frac{2.109}{4.528}}$$

$$= 0.682$$

Column 10—N_0

The expected crash frequency based on the assumption that different roadway elements are statistically independent, N_0, is calculated using Equation A-11 as follows:

$$N_0 = w_0 \times N_{\text{predicted (total)}} + (1 - w_0) \times N_{\text{observed (total)}}$$

$$= 0.682 \times 4.528 + (1 - 0.682) \times 9 \quad = 5.950$$

Column 11—w_1

The weight placed on predicted crash frequency under the assumption that crashes frequencies for different roadway elements are perfectly correlated, w_1, is calculated using Equation A-12 as follows:

$$w_1 = \frac{1}{1 + \frac{N_{\text{predicted } w1}}{N_{\text{predicted (total)}}}}$$

$$= \frac{1}{1 + \frac{2.076}{4.528}}$$

$$= 0.686$$

Column 12—N_1

The expected crash frequency based on the assumption that different roadway elements are perfectly correlated, N_1, is calculated using Equation A-13 as follows:

$$N_1 = w_1 \times N_{\text{predicted (total)}} + (1 - w_1) \times N_{\text{observed (total)}}$$

$$= 0.686 \times 4.528 + (1 - 0.686) \times 9 \quad = 5.932$$

Column 13—$N_{expected/comb}$

The expected average crash frequency based of combined sites, $N_{\text{expected}/comb}$, is calculated using Equation A-14 as follows:

$$N_{\text{expected}/comb} = \frac{N_0 + N_1}{2}$$

$$= \frac{5.950 + 5.932}{2}$$

$$= 5.941$$

Worksheet SP5B—Project-Level EB Method Summary Results for Rural Two-Lane, Two-Way Roads and Multilane Highways

Worksheet SP5B presents a summary of the results. The expected average crash frequency by severity level is calculated by applying the proportion of predicted average crash frequency by severity level to the total expected average crash frequency (Column 3).

Worksheet SP5B. Project-Level EB Method Summary Results for Rural Two-Lane, Two-Way Roads and Multilane Highways

(1)	(2)	(3)
Crash Severity Level	$N_{predicted}$	$N_{expected}$
Total	$(2)_{comb}$ from Worksheet SP5A	$(13)_{comb}$ from Worksheet SP5A
	4.528	5.9
Fatal and injury (FI)	$(3)_{comb}$ from Worksheet SP5A	$(3)_{total}*(2)_{FI}/(2)_{total}$
	2.299	3.0
Property damage only (PDO)	$(4)_{comb}$ from Worksheet SP5A	$(3)_{total}*(2)_{PDO}/(2)_{total}$
	2.229	2.9

11.12.6. Sample Problem 6

The Project

An existing rural two-lane roadway is proposed for widening to a four-lane highway facility. One portion of the project is planned as a four-lane divided highway, while another portion is planned as a four-lane undivided highway. There is one three-leg stop-controlled intersection located within the project limits.

The Question

What is the expected average crash frequency of the proposed rural four-lane highway facility for a particular year, and what crash reduction is expected in comparison to the existing rural two-lane highway facility?

The Facts

- Existing rural two-lane roadway facility with two roadway segments and one intersection equivalent to the facilities in Chapter 10's Sample Problems 1, 2, and 3.
- Proposed rural four-lane highway facility with two roadway segments and one intersection equivalent to the facilities in Sample Problems 1, 2, and 3 presented in this chapter.

Outline of Solution

Sample Problem 6 applies the Project Estimation Method 1 presented in Section C.7 (i.e., the expected average crash frequency for existing conditions is compared to the predicted average crash frequency of proposed conditions). The expected average crash frequency for the existing rural two-lane roadway can be represented by the results from applying the site-specific EB Method in Chapter 10's Sample Problem 5. The predicted average crash frequency for the proposed four-lane facility can be determined from the results of Sample Problems 1, 2, and 3 in this chapter. In this case, Sample Problems 1 through 3 are considered to represent a proposed facility rather than an existing facility; therefore, there is no observed crash frequency data, and the EB Method is not applicable.

Results

The predicted average crash frequency for the proposed four-lane facility project is 4.5 crashes per year, and the predicted crash reduction from the project is 7.8 crashes per year. Table 11-26 presents a summary of the results.

Table 11-26. Summary of Results for Sample Problem 6

Site	Expected Average Crash Frequency for the Existing Condition (crashes/year)[a]	Predicted Average Crash Frequency for the Proposed Condition (crashes/year)[b]	Predicted Crash Reduction from Project Implementation (crashes/year)
Segment 1	8.02	3.3	4.7
Segment 2	1.34	0.3	1.1
Intersection 1	2.94	0.9	2.0
Total	12.3	4.5	7.8

[a] From Sample Problems 5 in Chapter 10
[b] From Sample Problems 1 through 3 in Chapter 11

11.13. REFERENCES

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- Length of roadway segment (miles)
- AADT (vehicles per day)
- Number of through lanes
- Presence/type of median (undivided, divided by raised or depressed median, center TWLTL)
- Presence/type of on-street parking (parallel vs. angle; one side vs. both sides of street)
- Number of driveways for each driveway type (major commercial, minor commercial; major industrial/institutional; minor industrial/institutional; major residential; minor residential; other)
- Roadside fixed object density (fixed objects/mile, only obstacles 4-in or more in diameter that do not have a breakaway design are counted)
- Average offset to roadside fixed objects from edge of traveled way (feet)
- Presence/absence of roadway lighting
- Speed category (based on actual traffic speed or posted speed limit)
- Presence of automated speed enforcement

For all intersections within the study area, the following geometric and traffic control features are identified:

- Number of intersection legs (3 or 4)
- Type of traffic control (minor-road stop or signal)
- Number of approaches with intersection left-turn lane (all approaches, 0, 1, 2, 3, or 4 for signalized intersection; only major approaches, 0, 1, or 2, for stop-controlled intersections)
- Number of approaches with left-turn signal phasing (0, 1, 2, 3, or 4) (signalized intersections only) and type of left-turn signal phasing (permissive, protected/permissive, permissive/protected, or protected)
- Number of approaches with intersection right turn lane (all approaches, 0, 1, 2, 3, or 4 for signalized intersection; only major approaches, 0, 1, or 2, for stop-controlled intersections)
- Number of approaches with right-turn-on-red operation prohibited (0, 1, 2, 3, or 4) (signalized intersections only)
- Presence/absence of intersection lighting
- Maximum number of traffic lanes to be crossed by a pedestrian in any crossing maneuver at the intersection considering the presence of refuge islands (for signalized intersections only)
- Proportions of nighttime crashes for unlighted intersections (by total, fatal, injury, and property damage only)

For signalized intersections, land use and demographic data used in the estimation of vehicle-pedestrian collisions include:

- Number of bus stops within 1,000 feet of the intersection
- Presence of schools within 1,000 feet of the intersection
- Number of alcohol sales establishments within 1,000 feet of the intersection
- Presence of red light camera
- Number of approaches on which right-turn-on-red is allowed
- Pedestrian volumes

traffic control features). The base conditions for each SPF are specified in Section 12.6. A detailed explanation and overview of the SPFs are provided in Section C.6.3.

The SPFs developed for Chapter 12 are summarized in Table 12-2. For the selected site, determine the appropriate SPF for the site type (intersection or roadway segment) and the geometric and traffic control features (undivided roadway, divided roadway, stop-controlled intersection, signalized intersection). The SPF for the selected site is calculated using the AADT determined in Step 3 ($AADT_{maj}$ and $AADT_{min}$ for intersections) for the selected year.

Each SPF determined in Step 9 is provided with default distributions of crash severity and collision type (presented in Section 12.6). These default distributions can benefit from being updated based on local data as part of the calibration process presented in Part C, Appendix A.1.1.

Step 10—Multiply the result obtained in Step 9 by the appropriate CMFs to adjust base conditions to site specific geometric design and traffic control features.
In order to account for differences between the base conditions (Section 12.6) and the specific conditions of the site, CMFs are used to adjust the SPF estimate. An overview of CMFs and guidance for their use is provided in Section C.6.4, including the limitations of current knowledge related to the effects of simultaneous application of multiple CMFs. In using multiple CMFs, engineering judgment is required to assess the interrelationships and/or independence of individual elements or treatments being considered for implementation within the same project.

All CMFs used in Chapter 12 have the same base conditions as the SPFs used in Chapter 12 (i.e., when the specific site has the same condition as the SPF base condition, the CMF value for that condition is 1.00). Only the CMFs presented in Section 12.7 may be used as part of the Chapter 12 predictive method. Table 12-18 indicates which CMFs are applicable to the SPFs in Section 12.6.

The CMFs for roadway segments are those described in Section 12.7.1. These CMFs are applied as shown in Equation 12-3.

The CMFs for intersections are those described in Section 12.7.2, which apply to both signalized and stop-controlled intersections, and in Section 12.7.3, which apply to signalized intersections only. These CMFs are applied as shown in Equations 12-6 and 12-28.

In Chapter 12, the multiple- and single-vehicle base crashes determined in Step 9 and the CMFs values calculated in Step 10 are then used to estimate the vehicle-pedestrian and vehicle-bicycle base crashes for roadway segments and intersections (present in Sections 12.6.1 and 12.6.2 respectively).

Step 11—Multiply the result obtained in Step 10 by the appropriate calibration factor.
The SPFs used in the predictive method have each been developed with data from specific jurisdictions and time periods. Calibration to local conditions will account for these differences. A calibration factor (C_r for roadway segments or C_i for intersections) is applied to each SPF in the predictive method. An overview of the use of calibration factors is provided in Section C.6.5. Detailed guidance for the development of calibration factors is included in Part C, Appendix A.1.1.

Steps 9, 10, and 11 together implement the predictive models in Equations 12-2 through 12-7 to determine predicted average crash frequency.

Step 12—If there is another year to be evaluated in the study period for the selected site, return to Step 8. Otherwise, proceed to Step 13.
This step creates a loop through Steps 8 to 12 that is repeated for each year of the evaluation period for the selected site.

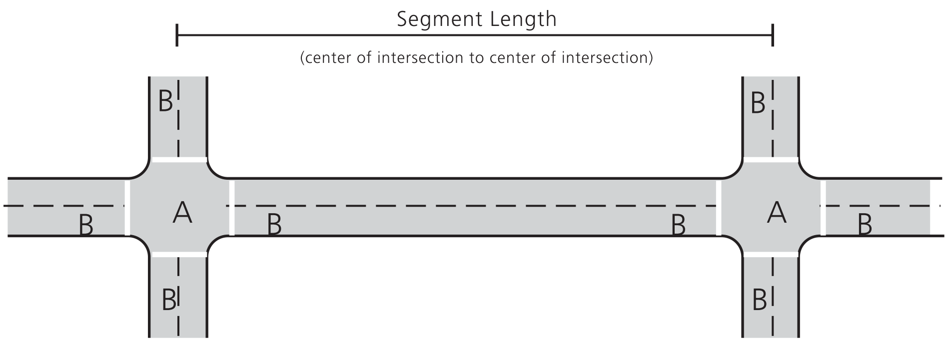

Figure 12-2. Definition of Roadway Segments and Intersections

The segmentation process produces a set of roadway segments of varying length, each of which is homogeneous with respect to characteristics such as traffic volumes and key roadway design characteristics and traffic control features. Figure 12-2 shows the segment length, *L*, for a single homogenous roadway segment occurring between two intersections. However, several homogenous roadway segments can occur between two intersections. A new (unique) homogeneous segment begins at the center of each intersection and where there is a change in at least one of the following characteristics of the roadway:

- Annual average daily traffic volume (AADT) (vehicles/day)
- Number of through lanes
- Presence/type of median
- Presence of TWLTL

The following rounded widths for medians without barriers are recommended before determining "homogeneous" segments:

Measured Median Width	Rounded Median Width
1 ft to 14 ft	10 ft
15 ft to 24 ft	20 ft
25 ft to 34 ft	30 ft
35 ft to 44 ft	40 ft
45 ft to 54 ft	50 ft
55 ft to 64 ft	60 ft
65 ft to 74 ft	70 ft
75 ft to 84 ft	80 ft
85 ft to 94 ft	90 ft
95 ft or more	100 ft

- Presence/type of on-street parking
- Roadside fixed object density
- Presence of lighting
- Speed category (based on actual traffic speed or posted speed limit)
- Automated enforcement

Table 12-9 presents the values of fbiker for use in Equation 12-18. All vehicle-bicycle collisions are considered to be fatal-and-injury crashes. The values of fbiker are likely to depend on the climate and bicycling environment in particular states or communities. HSM users are encouraged to replace the values in Table 12-9 with suitable values for their own state or community through the calibration process (see Part C, Appendix A).

Table 12-9. Bicycle Crash Adjustment Factors for Roadway Segments

	Bicycle Crash Adjustment Factor (f_{biker})	
Road type	**Posted Speed 30 mph or Lower**	**Posted Speed Greater than 30 mph**
2U	0.018	0.004
3T	0.027	0.007
4U	0.011	0.002
4D	0.013	0.005
5T	0.050	0.012

Note: These factors apply to the methodology for predicting total crashes (all severity levels combined).
All bicycle collisions resulting from this adjustment factor are treated as fatal-and-injury crashes and none as property-damage-only crashes.
Source: HSIS data for Washington (2002–2006)

12.6.2. Safety Performance Functions for Urban and Suburban Arterial Intersections

The predictive models for predicting the frequency of crashes related to an intersection is presented in Equations 12-5 through 12-7. The structure of the predictive models for intersections is similar to the predictive models for roadway segments.

The effect of traffic volume on predicted crash frequency for intersections is incorporated through SPFs, while the effect of geometric and traffic control features are incorporated through CMFs. Each of the SPFs for intersections incorporates separate effects for the AADTs on the major- and minor-road legs, respectively.

SPFs and adjustment factors have been developed for four types of intersections on urban and suburban arterials. These are:

- Three-leg intersections with stop control on the minor-road approach (3ST)
- Three-leg signalized intersections (3SG)
- Four-leg intersections with stop control on the minor-road approaches (4ST)
- Four-leg signalized intersections (4SG)

Other types of intersections may be found on urban and suburban arterials but are not addressed by the Chapter 12 SPFs.

The SPFs for each of the four intersection types identified above predict total crash frequency per year for crashes that occur within the limits of the intersection and intersection-related crashes. The SPFs and adjustment factors address the following four types of collisions, (the corresponding equations, tables, and figures are indicated in Table 12-2):

- multiple-vehicle collisions
- single-vehicle crashes
- vehicle-pedestrian collisions
- vehicle-bicycle collisions

Where:

CMF_{2r} = crash modification factor for the effect of roadside fixed objects on total crashes;

f_{offset} = fixed-object offset factor from Table 12-20;

D_{fo} = fixed-object density (fixed objects/mi) for both sides of the road combined; and

p_{fo} = fixed-object collisions as a proportion of total crashes from Table 12-21.

This CMF applies to total roadway segment crashes. If the computed value of CMF_{2r} is less than 1.00, it is set equal to 1.00. This can only occur for very low fixed object densities.

In estimating the density of fixed objects (D_{fo}), only point objects that are 4 inches or more in diameter and do not have breakaway design are considered. Point objects that are within 70 ft of one another longitudinally along the road are counted as a single object. Continuous objects that are not behind point objects are counted as one point object for each 70 ft of length. The offset distance (O_{fo}) shown in Table 12-20 is an estimate of the average distance from the edge of the traveled way to roadside objects over an extended roadway segment. If the average offset to fixed objects exceeds 30 ft, use the value of foffset for 30 ft. Only fixed objects on the roadside on the right side of the roadway in each direction of travel are considered; fixed objects in the roadway median on divided arterials are not considered.

Table 12-20. Fixed-Object Offset Factor

Offset to Fixed Objects (O_{fo}) (ft)	Fixed-Object Offset Factor (f_{offset})
2	0.232
5	0.133
10	0.087
15	0.068
20	0.057
25	0.049
30	0.044

Table 12-21. Proportion of Fixed-Object Collisions

Road Type	Proportion of Fixed-Object Collisions (p_{fo})
2U	0.059
3T	0.034
4U	0.037
4D	0.036
5T	0.016

CMF_{3r}—Median Width

A CMF for median widths on divided roadway segments of urban and suburban arterials is presented in Table 12-22 based on the work of Harkey et al. (6). The base condition for this CMF is a median width of 15 ft. The CMF applies to total crashes and represents the effect of median width in reducing cross-median collisions; the CMF assumes that nonintersection collision types other than cross-median collisions are not affected by median width. The CMF in Table 12-22 has been adapted from the CMF in Table 13-12 based on the estimate by Harkey et al. (6) that cross-median collisions represent 12.0 percent of crashes on divided arterials.

CMF_{5r}—Automated Speed Enforcement

Automated speed enforcement systems use video or photographic identification in conjunction with radar or lasers to detect speeding drivers. These systems automatically record vehicle identification information without the need for police officers at the scene. The base condition for automated speed enforcement is that it is absent. Chapter 17 presents a CMF of 0.83 for the reduction of all types of fatal-and-injury crashes from implementation of automated speed enforcement. This CMF is assumed to apply to roadway segments between intersections with fixed camera sites where the camera is always present or where drivers have no way of knowing whether the camera is present or not. No information is available on the effect of automated speed enforcement on noninjury crashes. With the conservative assumption that automated speed enforcement has no effect on noninjury crashes, the value of the CMF for automated speed enforcement would be 0.95.

12.7.2. Crash Modification Factors for Intersections

The effects of individual geometric design and traffic control features of intersections are represented in the predictive models by CMFs. CMF_{1i} through CMF_{6i} are applied to multiple-vehicle collisions and single-vehicle crashes at intersections, but not to vehicle-pedestrian and vehicle-bicycle collisions. CMF_{1p} through CMF_{3p} are applied to vehicle-pedestrian collisions at four-leg signalized intersections (4SG), but not to multiple-vehicle collisions and single-vehicle crashes and not to other intersection types.

CMF_{1i}—Intersection Left-Turn Lanes

The base condition for intersection left-turn lanes is the absence of left-turn lanes on the intersection approaches. The CMFs for presence of left-turn lanes are presented in Table 12-24. These CMFs apply to installation of left-turn lanes on any approach to a signalized intersection but only on uncontrolled major-road approaches to stop-controlled intersections. The CMFs for installation of left-turn lanes on multiple approaches to an intersection are equal to the corresponding CMF for installation of a left-turn lane on one approach raised to a power equal to the number of approaches with left-turn lanes. There is no indication of any change in crash frequency for providing a left-turn lane on an approach controlled by a stop sign, so the presence of a left-turn lane on a stop-controlled approach is not considered in applying Table 12-24. The CMFs in the table apply to total intersection crashes (not including vehicle-pedestrian and vehicle-bicycle collisions). The CMFs for installation of left-turn lanes are based on research by Harwood et al. (7). A CMF of 1.00 is always used when no left-turn lanes are present.

Table 12-24. Crash Modification Factor (CMF_{1i}) for Installation of Left-Turn Lanes on Intersection Approaches

		Number of Approaches with Left-Turn Lanes[a]			
Intersection Type	**Intersection Traffic Control**	**One Approach**	**Two Approaches**	**Three Approaches**	**Four Approaches**
Three-leg intersection	Minor-road stop control[b]	0.67	0.45	—	—
	Traffic signal	0.93	0.86	0.80	—
Four-leg intersection	Minor-road stop control[b]	0.73	0.53	—	—
	Traffic signal	0.90	0.81	0.73	0.66

[a] Stop-controlled approaches are not considered in determining the number of approaches with left-turn lanes.
[b] Stop signs present on minor-road approaches only.

CMF_{2i}—Intersection Left-Turn Signal Phasing

The CMF for left-turn signal phasing is based on the results of work by Hauer (10), as modified in a study by Lyon et al. (11). Types of left-turn signal phasing considered include permissive, protected, protected/permissive, and permissive/protected. Protected/permissive operation is also referred to as a leading left-turn signal phase; permissive/protected operation is also referred to as a lagging left-turn signal phase. The CMF values are presented in Table 12-25. The base condition for this CMF is permissive left-turn signal phasing. This CMF applies to total intersection crashes (not including vehicle-pedestrian and vehicle-bicycle collisions) and is applicable only to signalized intersections. A CMF value of 1.00 is always used for unsignalized intersections.

If several approaches to a signalized intersection have left-turn phasing, the values of CMF_{2i} for each approach are multiplied together.

12.13.4. Sample Problem 4

The Intersection

A four-leg signalized intersection located on an urban arterial.

The Question

What is the predicted crash frequency of the signalized intersection for a particular year?

The Facts

- 1 left-turn lane on each of the two major road approaches
- 1 right-turn lane on each of the two major road approaches
- Protected/permissive left-turn signal phasing on major road
- AADT of major road is 15,000 veh/day
- AADT of minor road is 9,000 veh/day
- Lighting is present
- No approaches with prohibited right-turn-on-red
- Four-lane divided major road
- Two-lane undivided minor road
- Pedestrian volume is 1,500 peds/day
- The number of bus stops within 1,000 ft of intersection is 2
- A school is present within 1,000 ft of intersection
- The number of alcohol establishments within 1,000 ft of intersection is 6

Assumptions

Collision type distributions used are the default values from Tables 12-11 and 12-13 and Equations 12-28 and 12-31.

The calibration factor is assumed to be 1.00.

The maximum number of lanes crossed by a pedestrian is assumed to be four (crossing two through lanes, one left-turn lane, and one right-turn lane across one side of the divided major road).

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the signalized intersection in Sample Problem 4 is determined to be 3.4 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8

To determine the predicted average crash frequency of the roadway segment in Sample Problem 4, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year and the EB Method is not applied.

Worksheet 2B. Crash Modification Factors for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
CMF for Left-Turn Lanes	**CMF for Left-Turn Signal Phasing**	**CMF for Right-Turn Lanes**	**CMF for Right-Turn-on-Red**	**CMF for Lighting**	**CMF for Red-Light Cameras**	**Combined CMF**
CMF_{1i}	CMF_{2i}	CMF_{3i}	CMF_{4i}	CMF_{5i}	CMF_{6i}	CMF_{comb}
from Table 12-24	from Table 12-25	from Table 12-26	from Equation 12-35	from Equation 12-36	from Equation 12-37	(1)*(2)*(3)*(4)*(5)*(6)

Worksheet 2C. Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)
	SPF Coefficients			**Overdispersion Parameter, *k***	**Initial N_{bimv}**
	from Table 12-10				
Crash Severity Level	**a**	**b**	**c**	**from Table 12-10**	**from Equation 12-22**
Total					
Fatal and injury (FI)					
Property damage only (PDO)					

Worksheet 2C. Continued

(1)	(5)	(6)	(7)	(8)	(9)
		Adjusted N_{bimv}	**Combined CMFs**		**Predicted N_{bimv}**
Crash Severity Level	**Proportion of Total Crashes**	**$(4)_{total}$*(5)**	**(7) from Worksheet 2B**	**Calibration Factor, C_i**	**(6)*(7)*(8)**
Total					
Fatal and injury (FI)	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$				
Property damage only (PDO)	$(5)_{total}-(5)_{FI}$				

Worksheet 2D. Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type $_{(FI)}$	**Predicted $N_{bimv\ (FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**Predicted $N_{bimv\ (PDO)}$ (crashes/year)**	**Predicted $N_{bimv\ (total)}$ (crashes/year)**
Collision Type	from Table 12-11	$(9)_{FI}$ from Worksheet 2C	from Table 12-11	$(9)_{PDO}$ from Worksheet 2C	$(9)_{PDO}$ from Worksheet 2C
Total	1.000	$(2)*(3)_{FI}$	1.000	$(4)*(5)_{PDO}$	(3)+(5)
Rear-end collision					
Head-on collision					
Angle collision					
Sideswipe					
Other multiple-vehicle collision					

Worksheet 2E. Single-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)
	SPF Coefficients			Overdispersion Parameter, k	Initial N_{bisv}
	from Table 12-12				
Crash Severity Level	a	b	c	from Table 12-12	from Equation 12-25; (FI) from Equation 12-25 or 12-27
Total					
Fatal and injury (FI)					
Property damage only (PDO)					

Worksheet 2E. Continued

(1)	(5)	(6)	(7)	(8)	(9)
		Adjusted N_{bisv}	Combined CMFs		Predicted N_{bisv}
Crash Severity Level	Proportion of Total Crashes	$(4)_{total}$*(5)	(7) from Worksheet 2B	Calibration Factor, C_i	(6)*(7)*(8)
Total					
Fatal and injury (FI)	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$				
Property damage only (PDO)	$(5)_{total}-(5)_{FI}$				

Worksheet 2F. Single-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
	Proportion of Collision Type $_{(FI)}$	Predicted $N_{bisv\,(FI)}$ (crashes/year)	Proportion of Collision Type $_{(PDO)}$	Predicted $N_{bisv\,(PDO)}$ (crashes/year)	Predicted $N_{bisv\,(total)}$ (crashes/year)
Collision Type	Table 12-13	$(9)_{FI}$ from Worksheet 2E	Table 12-13	$(9)_{PDO}$ from Worksheet 2E	$(9)_{PDO}$ from Worksheet 2E
Total	1.000	$(2)*(3)_{FI}$	1.000	$(4)*(5)_{PDO}$	(3)+(5)
Collision with parked vehicle					
Collision with animal					
Collision with fixed object					
Collision with other object					
Other single-vehicle collision					
Single-vehicle noncollision					

Worksheet 2G. Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Stop-Controlled Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Predicted N_{bimv}	Predicted N_{bisv}	Predicted N_{bi}	f_{pedi}		Predicted N_{pedi}
Crash Severity Level	(9) from Worksheet 2C	(9) from Worksheet 2E	(2)+(3)	from Table 12-16	Calibration Factor, C_i	(4)*(5)*(6)
Total						
Fatal and injury (FI)	—	—	—	—		

Table A-2. Data Needs for Calibration of Part C Predictive Models by Facility Type

Chapter	Data Element	Data Need: Required	Data Need: Desirable	Default Assumption
ROADWAY SEGMENTS				
10—Rural Two-Lane, Two-Way Roads	Segment length	X		Need actual data
	Annual average daily traffic (AADT)	X		Need actual data
	Lengths of horizontal curves and tangents	X		Need actual data
	Radii of horizontal curves	X		Need actual data
	Presence of spiral transition for horizontal curves		X	Base default on agency design policy
	Superelevation variance for horizontal curves		X	No superelevation variance
	Percent grade		X	Base default on terrain[a]
	Lane width	X		Need actual data
	Shoulder type	X		Need actual data
	Shoulder width	X		Need actual data
	Presence of lighting		X	Assume no lighting
	Driveway density		X	Assume 5 driveways per mile
	Presence of passing lane		X	Assume not present
	Presence of short four-lane section		X	Assume not present
	Presence of center two-way left-turn lane	X		Need actual data
	Presence of centerline rumble strip		X	Base default on agency design policy
	Roadside hazard rating		X	Assume roadside hazard rating = 3
	Use of automated speed enforcement		X	Base default on current practice
11—Rural Multilane Highways	*For all rural multilane highways*:			
	Segment length	X		Need actual data
	Annual average daily traffic (AADT)	X		Need actual data
	Lane width	X		Need actual data
	Shoulder width	X		Need actual data
	Presence of lighting		X	Assume no lighting
	Use of automated speed enforcement		X	Base default on current practice
	For undivided highways only:			
	Sideslope	X		Need actual data
	For divided highways only:			
	Median width	X		Need actual data

Table A-2. Continued on next page

Table A-2. Data Needs for Calibration of Part C Predictive Models by Facility Type *continued*

Chapter	Data Element	Data Need: Required	Data Need: Desirable	Default Assumption
12—Urban and Suburban Arterials	Segment length	X		Need actual data
	Number of through traffic lanes	X		Need actual data
	Presence of median	X		Need actual data
	Presence of center two-way left-turn lane	X		Need actual data
	Annual average daily traffic (AADT)	X		Need actual data
	Number of driveways by land-use type	X		Need actual data[b]
	Posted speed limit	X		Need actual data
	Presence of on-street parking	X		Need actual data
	Type of on-street parking	X		Need actual data
	Roadside fixed object density		X	database default on fixed-object offset and density categories[c]
	Presence of lighting		X	Base default on agency practice
	Presence of automated speed enforcement		X	Base default on agency practice
INTERSECTIONS				
10—Rural Two-Lane, Two-Way Roads	Number of intersection legs	X		Need actual data
	Type of traffic control	X		Need actual data
	Annual average daily traffic (AADT) for major road	X		Need actual data
	Annual average daily traffic (AADT) for minor road	X		Need actual data or best estimate
	Intersection skew angle		X	Assume no skew[d]
	Number of approaches with left-turn lanes	X		Need actual data
	Number of approaches with right-turn lanes	X		Need actual data
	Presence of lighting	X		Need actual data
11—Rural Multilane Highways	*For all rural multilane highways:*			
	Number of intersection legs	X		Need actual data
	Type of traffic control	X		Need actual data
	Annual average daily traffic (AADT) for major road	X		Need actual data
	Annual average daily traffic (AADT) for minor road	X		Need actual data or best estimate
	Presence of lighting	X		Need actual data
	Intersection skew angle		X	Assume no skew[d]
	Number of approaches with left-turn lanes	X		Need actual data
	Number of approaches with right-turn lanes	X		Need actual data

Table A-2. Continued on next page

Table A-2. Data Needs for Calibration of Part C Predictive Models by Facility Type *continued*

Chapter	Data Element	Data Need: Required	Data Need: Desirable	Default Assumption
12—Urban and Suburban Arterials	*For all intersections on arterials:*			
	Number of intersection legs	X		Need actual data
	Type of traffic control	X		Need actual data
	Average annual daily traffic (AADT) for major road	X		Need actual data
	Average annual daily traffic (AADT) for minor road	X		Need actual data or best estimate
	Number of approaches with left-turn lanes	X		Need actual data
	Number of approaches with right-turn lanes	X		Need actual data
	Presence of lighting	X		Need actual data
	For signalized intersections only:			
	Presence of left-turn phasing	X		Need actual data
	Type of left-turn phasing	X		Prefer actual data, but agency practice may be used as a default
	Use of right-turn-on-red signal operation	X		Need actual data
	Use of red-light cameras	X		Need actual data
	Pedestrian volume		X	Estimate with Table 12-15
	Maximum number of lanes crossed by pedestrians on any approach		X	Estimate from number of lanes and presence of median on major road
	Presence of bus stops within 1,000 ft		X	Assume not present
	Presence of schools within 1,000 ft		X	Assume not present
	Presence of alcohol sales establishments within 1,000 ft		X	Assume not present

[a] Suggested default values for calibration purposes: *CMF* = 1.00 for level terrain; *CMF* = 1.06 for rolling terrain; *CMF* = 1.14 for mountainous terrain
[b] Use actual data for number of driveways, but simplified land-use categories may be used (e.g., commercial and residential only).
[c] CMFs may be estimated based on two categories of fixed-object offset (O_{fo})—either 5 or 20 ft—and three categories of fixed-object density (D_{fo})—0, 50, or 100 objects per mile.
[d] If measurements of intersection skew angles are not available, the calibration should preferably be performed for intersections with no skew.

A.1.1.4. Step 4—Apply the Applicable Part C Predictive Method to Predict Total Crash Frequency for Each Site During the Calibration Period as a Whole

The site characteristics data assembled in Step 3 should be used to apply the applicable predictive method from Chapter 10, 11, or 12 to each site in the calibration data set. For this application, the predictive method should be applied without using the EB Method and, of course, without employing a calibration factor (i.e., a calibration factor of 1.00 is assumed). Using the predictive models, the expected average crash frequency is obtained for either one, two, or three years, depending on the duration of the calibration period selected.

A.1.1.5. Step 5—Compute Calibration Factors for Use in Part C Predictive Models

The final step is to compute the calibration factor as:

$$C_r(\text{or } C_i) = \frac{\sum_{\text{all sites}} \text{observed crashes}}{\sum_{\text{all sites}} \text{predicted crashes}} \tag{A-1}$$

The computation is performed separately for each facility type. The computed calibration factor is rounded to two decimal places for application in the appropriate Part C predictive model.

Example Calibration Factor Calculation

The SPF for four-leg signalized intersections on rural two-lane, two-way roads from Equation 10-10 is:

$$N_{spf\ int} = e^{[-5.13 + 0.60 \times \ln(AADT_{maj}) + 0.20 \times \ln(AADT_{min})]}$$

Where:

$N_{spf\ int}$ = predicted number of total intersection-related crashes per year for base conditions;

$AADT_{maj}$ = average annual daily entering traffic volumes (vehicles/day) on the major road; and

$AADT_{min}$ = average annual daily entering traffic volumes (vehicles/day) on the minor road.

The base conditions are:

- No left-turn lanes on any approach
- No right-turn lanes on any approach

The CMF values from Chapter 10 are:

- CMF for one approach with a left-turn lane = 0.82
- CMF for one approach with a right-turn lane = 0.96
- CMF for two approaches with right-turn lanes = 0.92
- No lighting present (so lighting CMF = 1.00 for all cases)

Typical data for eight intersections is shown in an example calculation shown below. Note that for an actual calibration, the recommended minimum sample size would be 30 to 50 sites that experience at least 100 crashes per year. Thus, the number of sites used here is smaller than recommended, and is intended solely to illustrate the calculations.

For the first intersection in the example the predicted crash frequency for base conditions is:

$$N_{bibase} = e^{(-5.13 + 0.60 \times \ln(4000) + 0.20 \times \ln(2000))} = 3.922 \text{ crashes/year}$$

The intersection has a left-turn lane on the major road, for which CMF_{1i} is 0.82, and a right-turn lane on one approach, a feature for which CMF_{2i} is 0.96. There are three years of data, during which four crashes were observed (shown in Column 10 of Table Ex-1). The predicted average crash frequency from the Chapter 10 for this intersection without calibration is from Equation 10-2:

$$N_{bi} = (N_{bibase}) \times (CMF_{1i}) \times (CMF_{2i}) \times (\text{number of years of data})$$

$$= 3.922 \times 0.82 \times 0.96 \times 3 = 9.262 \text{ crashes in three years, shown in Column 9.}$$

Similar calculations were done for each intersection in the table shown below. The sum of the observed crash frequencies in Column 10 (43) is divided by the sum of the predicted average crash frequencies in Column 9 (87.928) to obtain the calibration factor, C_i, equal to 0.489. It is recommended that calibration factors be rounded to two decimal places, so calibration factor equal to 0.49 should be used in the Chapter 10 predictive model for four-leg signalized intersections.

Table Ex-1. Example of Calibration Factor Computation

1	2	3	4	5	6	7	8	9	10
$AADT_{maj}$	$AADT_{min}$	SPF Prediction	Intersection Approaches with Left-Turn Lanes	CMF_{1i}	Intersection Approaches with Right-Turn Lane	CMF_{2i}	Years of Data	Predicted Average Crash Frequency	Observed Crash Frequency
4000	2000	3.922	1	0.82	1	0.96	3	9.262	4
3000	1500	3.116	0	1.00	2	0.92	2	5.733	5
5000	3400	4.986	0	1.00	2	0.92	3	13.761	10
6500	3000	5.692	0	1.00	2	0.92	3	15.709	5
3600	2300	3.786	1	0.82	1	0.96	3	8.941	2
4600	4500	5.016	0	1.00	2	0.92	3	13.844	8
5700	3300	5.362	1	0.82	1	0.96	3	12.662	5
6800	1500	5.091	1	0.82	1	0.96	2	8.015	4
							Sum	87.928	43
							Calibration Factor (C_i)		0.489

A.1.2. Development of Jurisdiction-Specific Safety Performance Functions for Use in the Part C Predictive Method

Satisfactory results from the Part C predictive method can be obtained by calibrating the predictive model for each facility type, as explained in Appendix A.1.1. However, some users may prefer to develop jurisdiction-specific SPFs using their agency's own data, and this is likely to enhance the reliability of the Part C predictive method. While there is no requirement that this be done, HSM users are welcome to use local data to develop their own SPFs, or if they wish, replace some SPFs with jurisdiction-specific models and retain other SPFs from the Part C chapters. Within the first two to three years after a jurisdiction-specific SPF is developed, calibration of the jurisdiction-specific SPF using the procedure presented in Appendix A.1.1 may not be necessary, particularly if other default values in the Part C models are replaced with locally-derived values, as explained in Appendix A.1.3.

If jurisdiction-specific SPFs are used in the Part C predictive method, they need to be developed with methods that are statistically valid and developed in such a manner that they fit into the applicable Part C predictive method. The following guidelines for development of jurisdiction-specific SPFs that are acceptable for use in Part C include:

- In preparing the crash data to be used for development of jurisdiction-specific SPFs, crashes are assigned to roadway segments and intersections following the definitions explained in Appendix A.2.3 and illustrated in Figure A-1.
- The jurisdiction-specific SPF should be developed with a statistical technique such as negative binomial regression that accounts for the overdispersion typically found in crash data and quantifies an overdispersion parameter so that the model's predictions can be combined with observed crash frequency data using the EB Method.
- The jurisdiction-specific SPF should use the same base conditions as the corresponding SPF in Part C or should be capable of being converted to those base conditions.
- The jurisdiction-specific SPF should include the effects of the following traffic volumes: average annual daily traffic volume for roadway segment and major- and minor-road average annual daily traffic volumes for intersections.
- The jurisdiction-specific SPF for any roadway segment facility type should have a functional form in which predicted average crash frequency is directly proportional to segment length.

Table A-3. Default Crash Distributions Used in Part C Predictive Models Which May Be Calibrated by Users to Local Conditions

Chapter	Table or Equation Number	Type of Roadway Element: Roadway Segments	Type of Roadway Element: Intersections	Data Element or Distribution That May Be Calibrated to Local Conditions
10—Rural Two-Lane, Two-Way Roads	Table 10-3	X		Crash severity by facility type for roadway segments
	Table 10-4	X		Collision type by facility type for roadway segments
	Table 10-5		X	Crash severity by facility type for intersections
	Table 10-6		X	Collision type by facility type for intersections
	Equation 10-18	X		Driveway-related crashes as a proportion of total crashes (p_{dwy})
	Table 10-12	X		Nighttime crashes as a proportion of total crashes by severity level
	Table 10-15		X	Nighttime crashes as a proportion of total crashes by severity level and by intersection type
11—Rural Multilane Highways	Table 11-4	X		Crash severity and collision type for undivided segments
	Table 11-6	X		Crash severity and collision type for divided segments
	Table 11-9		X	Crash severity and collision type by intersection type
	Table 11-15	X		Nighttime crashes as a proportion of total crashes by severity level and by roadway segment type for undivided roadway segments
	Table 11-19	X		Nighttime crashes as a proportion of total crashes by severity level and by roadway segment type for divided roadway segments
	Table 11-24		X	Nighttime crashes as a proportion of total crashes by severity level and by intersection type
12—Urban and Suburban Arterials	Table 12-4	X		Crash severity and collision type for multiple-vehicle nondriveway collisions by roadway segment type
	Table 12-6	X		Crash severity and collision type for single-vehicle crashes by roadway segment type
	Table 12-7	X		Crash severity for driveway-related collisions by roadway segment type[a]
	Table 12-8	X		Pedestrian crash adjustment factor by roadway segment type
	Table 12-9	X		Bicycle crash adjustment factor by roadway segment type
	Table 12-11		X	Crash severity and collision type for multiple-vehicle collisions by intersection type
	Table 12-13		X	Crash severity and collision type for single-vehicle crashes by intersection type
	Table 12-16		X	Pedestrian crash adjustment factor by intersection type for stop-controlled intersections
	Table 12-17		X	Bicycle crash adjustment factor by intersection type
	Table 12-23	X		Nighttime crashes as a proportion of total crashes by severity level and by roadway segment type
	Table 12-27		X	Nighttime crashes as a proportion of total crashes by severity level and by intersection type

[a] The only portion of Table 12-7 that should be modified by the user are the crash severity proportions.

Note: No quantitative values in the Part C predictive models, other than those listed here and those discussed in Appendices A.1.1 and A.1.2, should be modified by HSM users.

$$f_{pedr} = \frac{K_{ped}}{K_{non}} \tag{A-2}$$

Where:

f_{pedr} = pedestrian crash adjustment factor;

K_{ped} = observed vehicle-pedestrian crash frequency; and

K_{non} = observed frequency for all crashes not including vehicle-pedestrian and vehicle-bicycle crash.

The pedestrian crash adjustment factor for a given facility type should be determined with a set of sites of that speed type that, as a group, includes at least 20 vehicle-pedestrian collisions.

Bicycle Crash Adjustment Factor by Roadway Segment Type

Table 12-9 presents a bicycle crash adjustment factor for specific roadway segment facility types and for two speed categories: low speed (traffic speeds or posted speed limits of 30 mph or less) and intermediate or high speed (traffic speeds or posted speed limits greater than 30 mph). For a given facility type and speed category, the bicycle crash adjustment factor is computed as:

$$f_{biker} = \frac{K_{bike}}{K_{non}} \tag{A-3}$$

Where:

f_{biker} = bicycle crash adjustment factor;

K_{bike} = observed vehicle-bicycle crash frequency; and

K_{non} = observed frequency for all crashes not including vehicle-pedestrian and vehicle-bicycle crashes.

The bicycle crash adjustment factor for a given facility type should be determined with a set of sites of that speed type that, as a group, includes at least 20 vehicle-bicycle collisions.

Crash Severity and Collision Type for Multiple-Vehicle Crashes by Intersection Type

Table 12-11 presents the combined distribution of crashes for two crash severity levels and six collision types. If sufficient data are available for a given facility type, the values in Table 12-11 for that facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 crashes in the time period for which data are available.

Crash Severity and Collision Type for Single-Vehicle Crashes by Intersection Type

Table 12-13 presents the combined distribution of crashes for two crash severity levels and six collision types. If sufficient data are available for a given facility type, the values in Table 12-13 for that facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 crashes in the time period for which data are available. The default values for f_{bisv} in Equation 12-27 should be replaced with locally available data.

Pedestrian Crash Adjustment Factor by Intersection Type

Table 12-16 presents a pedestrian crash adjustment factor for two specific types of intersections with stop control on the minor road. For a given facility type and speed category, the pedestrian crash adjustment factor is computed using Equation A-2. The pedestrian crash adjustment factor for a given facility type is determined with a set of sites that, as a group, have experienced at least 20 vehicle-pedestrian collisions.

Bicycle Crash Adjustment Factor by Intersection Type

Table 12-17 presents a bicycle crash adjustment factor for four specific intersection facility types. For a given facility type, the bicycle crash adjustment factor is computed using Equation A-3. The bicycle crash should, therefore, be excluded from the computations with the EB Method. Chapter 12 uses multiple models with different overdispersion parameters in safety predictions for any specific roadway segment or intersection. Where observed crash data are aggregated so that the corresponding value of predicted crash frequency is determined as the sum of the results from multiple predictive models with differing overdispersion parameters, the project-level EB Method presented in Appendix A.2.5 should be applied rather than the site-specific method presented here.

Chapters 10, 11, and 12 each present worksheets that can be used to apply the site-specific EB Method as presented in this section.

Appendix A.2.6 explains how to update N_{expected} to a future time period, such as the time period when a proposed future project will be implemented. This procedure is only applicable if the conditions of the proposed project will not be substantially different from the roadway conditions during which the observed crash data was collected.

A.2.5. Apply the Project-Level EB Method

HSM users may not always have location specific information for observed crash data for the individual roadway segments and intersections that make up a facility or project of interest. Alternative procedures are available where observed crash frequency data are aggregated across several sites (e.g., for an entire facility or project). This requires a more complex EB Method for two reasons. First, the overdispersion parameter, k, in the denominator of Equation A-5 is not uniquely defined, because estimate of crash frequency from two or more predictive models with different overdispersion parameters are combined. Second, it cannot be assumed, as is normally done, that the expected average crash frequency for different site types are statistically correlated with one another. Rather, an estimate of expected average crash frequency should be computed based on the assumption that the various roadway segments and intersections are statistically independent ($r = 0$) and on the alternative assumption that they are perfectly correlated ($r = 1$). The expected average crash frequency is then estimated as the average of the estimates for $r = 0$ and $r = 1$.

The following equations implement this approach, summing the first three terms, which represent the three roadway-segment-related crash types, over the five types of roadway segments considered in the (2U, 3T, 4U, 4D, 5T) and the last two terms, which represent the two intersection-related crash types, over the four types of intersections (3ST, 3SG, 4ST, 4SG):

$$N_{\text{predicted (total)}} = \sum_{j=1}^{5} N_{\text{predicted } rmj} + \sum_{j=1}^{5} N_{\text{predicted } rsj} + \sum_{j=1}^{5} N_{\text{predicted } rdj} + \sum_{j=1}^{4} N_{\text{predicted } imj} + \sum_{j=1}^{4} N_{\text{predicted } isj} \tag{A-6}$$

$$N_{\text{observed (total)}} = \sum_{j=1}^{5} N_{\text{observed } rmj} + \sum_{j=1}^{5} N_{\text{observed } rsj} + \sum_{j=1}^{5} N_{\text{observed } rdj} + \sum_{j=1}^{4} N_{\text{observed } imj} + \sum_{j=1}^{4} N_{\text{observed } isj} \tag{A-7}$$

$$\begin{aligned} N_{\text{predicted } w0} = {} & \sum_{j=1}^{5} k_{rmj} N_{rmj}^2 + \sum_{j=1}^{5} k_{rsj} N_{rsj}^2 + \sum_{j=1}^{5} k_{rdj} N_{rdj}^2 + \\ & \sum_{j=1}^{4} k_{imj} N_{imj}^2 + \sum_{j=1}^{4} k_{isj} N_{isj}^2 \end{aligned} \tag{A-8}$$

$$N_{\text{predicted},w1} = \begin{bmatrix} \sum_{j=1}^{5}\sqrt{k_{rmj}N_{rmj}^2} + \sum_{j=1}^{5}\sqrt{k_{rsj}N_{rsj}^2} \\ +\sum_{j=1}^{5}\sqrt{k_{rdj}N_{rdj}^2} + \sum_{j=1}^{4}\sqrt{k_{imj}N_{imj}^2} \\ +\sum_{j=1}^{4}\sqrt{k_{isj}N_{isj}^2} \end{bmatrix}^2 \tag{A-9}$$

facilities under design and of existing facilities under extensive re-design. It facilitates a proactive approach to considering safety before crashes occur. Some Part D CMFs are included in Part C and for use with specific Safety Performance Functions (SPFs). Other Part D CMFs are not presented in Part C but can be used in the methods to estimate change in crash frequency described in Section C.7.

D.4. GUIDE TO APPLYING PART D

The notations and terms cited and defined in the subsections below are used to indicate the level of knowledge regarding the effects on crash frequency of the various geometric and operational elements presented throughout Part D.

The following subsections explain useful information about:

- How the CMFs are categorized and organized in each chapter;
- The notation used to convey the reliability of each CMF;
- Terminology used in each chapter;
- Application of CMFs; and
- Considerations when applying CMFs.

To effectively use the crash modification factors in Part D, it is important to understand the notations and terminology, as well as the situation in which the countermeasure associated with the CMF is going to be applied. Understanding these items will increase the likelihood of success when implementing countermeasures.

D.4.1. Categories of Information

At the beginning of each section of Part D, treatments are summarized in tables according to the category of information available (i.e., crash modification factors or evidence of trends). These tables serve as a quick reference of the information available related to a specific treatment. Table D-1 summarizes how the information is categorized.

Table D-1. Categories of Information in Part D

Symbol Used in Part D Summary Tables	Available Information
✓	CMFs are available (i.e., sufficient quantitative information is available to determine a reliable CMF). The CMFs and standard errors passed the screening test to be included in the HSM.
T	There is some evidence of the effects on crash frequency, although insufficient quantitative information is available to determine a reliable CMF. In some instances, the quantitative information is sufficient to identify a known trend or apparent trend in crash frequency and/or user behavior, but not sufficient to apply in estimating changes in crash frequency. Published documentation regarding the treatment was not sufficiently reliable to present a CMF in this edition of the HSM. A list of these treatments is presented in the appendices to each chapter.
—	Quantitative information about the effects on crash frequency is not available for this edition of the HSM. Published documentation did not include quantitative information regarding the effects on crash frequency of the treatment. A list of these treatments is presented in the appendices to each chapter.

For those treatments with CMFs, the CMFs and standard errors are provided in tables. When available, each table supplies the specific treatment, road type or intersection type, setting (i.e., rural, urban, or suburban), traffic volumes, and crash type and severity to which the CMF can be applied.

- ^: The carat indicates that the CMF value itself is within the range 0.90 to 1.10 but that the lower or upper end of the confidence interval (defined by the CMF ± two times the standard error) may be exactly at 1.0. This is important to note since a treatment with such a CMF may result in no change in safety. These CMFs should be used with caution.
- º: The degree symbol indicates that the standard error has not been quantified for the CMF; therefore, the potential error inherent in the value is not known. This usually occurs when the factor is included as an equation.
- +: The plus sign indicates that the CMF is the result of combining CMFs from multiple studies.
- ?: The question mark indicates CMFs that have the opposite effects on different crash types or crash severities. For example, a treatment may increase rear-end crashes but decrease angle crashes. Or a treatment may reduce fatal crashes but increase property damage only (PDO) crashes.

Understanding the meanings of the superscripts and the standard error of a CMF will build familiarity with the reliability and stability that can be expected from each treatment. A CMF with a relatively high standard error does not mean that it should not be used; it means that the CMF should be used with the awareness of the range of results that could be obtained. Applying these treatments is also an opportunity to study the effectiveness of the treatment after implementation and add to the current information available regarding the treatment's effectiveness (see Chapter 9, "Safety Effectiveness Evaluation" for more information).

D.4.3. Terminology

Described below are some of the key words used in Part D to describe the CMF values or information provided. Key words to understand are:

- Unspecified: In some cases, CMF tables include some characteristics that are "unspecified." This indicates that the research did not clearly state the road type or intersection type, setting, or traffic volumes of the study.
- Injury: In Part D of the HSM, injury crashes include fatal crashes unless otherwise noted.
- All Settings: In some instances, research presented aggregated results for multiple settings (e.g., urban and suburban signalized intersections); the same level of information is reflected in the HSM.
- Insufficient or No Quantitative Information Available: Indicates that the documentation reviewed for the HSM did not contain quantitative information that passed the screening test for inclusion in the HSM. It doesn't mean that such documentation does not exist.

D.4.4. Application of CMFs to Estimate Crash Frequency

As discussed above, CMFs are used to estimate crash frequency or the change in crashes due to a treatment. There are multiple approaches to calculate an estimated number of crashes using a CMF. These include:

1. Applying the CMF to an expected number of crashes calculated using a calibrated safety performance function and EB to account for regression-to-the-mean bias;

2. Applying the CMF to a predicted number of crashes calculated using a calibrated safety performance function; and

3. Applying the CMF to historic crash count data.

Of the three ways to apply CMFs, listed above, the first approach produces the most reliable results. The second approach is the second most reliable and the third approach is the approach used if a safety performance function is not available to calculate the expected number of crashes. Additional details regarding safety performance functions, expected number of crashes, regression-to-the-mean, and EB methodology are discussed in Chapter 3, "Fundamentals." The specific step-by-step process for calculating an estimated change in crashes using approach 1 or 2 listed above is presented in Chapter 7, "Economic Appraisal."

3) Calculate the difference between the expected number of crashes without the treatment and the expected number with the treatment.

Change in Expected Average Crash Frequency

a) For total crashes

30.0 – 25.5 = 4.5 crashes/year reduction

b) For single vehicle crashes

8.0 – 5.9 = 2.1 crashes/year reduction

4) Discussion: The change in sideslope from 1V:3H to 1V:7H may potentially cause a reduction of 4.5 total crashes/year and 2.1 single vehicle crashes/year. A standard error is not available for these CMFs.

Rural multilane highways

Table 13-20 presents CMFs for the effect of sideslopes on multilane undivided roadway segments. These CMFs were developed by Harkey et al. (10) from the work of Zeeger et al. (6). The base condition for this CMF (i.e., the condition in which the CMF = 1.00) is a sideslope of 1V:7H or flatter.

Table 13-20. Potential Crash Effects of Sideslopes on Undivided Segments (15,34)

Treatment	Setting (Road Type)	Traffic Volume	Crash Type (Severity)	CMF	Std. Error
1V:7H or Flatter	Rural (Multilane highway)	Unspecified	All types (Unspecified)	1.00	N/A
1V:6H				1.05	
1V:5H				1.09	
1V:4H				1.12	
1V:2H or Steeper				1.18	
Base Condition: Provision of a 1V:7H or flatter sideslope.					

13.5.2.2. Increase the Distance to Roadside Features

Rural two-lane roads and freeways

The crash effects of increasing the distance to roadside features from 3.3 ft to 16.7 ft, or from 16.7 ft to 30.0 ft are shown in Table 13-21 (8). CMF values for other increments may be interpolated from the values presented in Table 13-21.

The base condition of the CMFs (i.e., the condition in which the CMF = 1.00) is a distance of either 3.3 ft or 16.7 ft to roadside features depending on original geometry.

13.5.2.4. Install Median Barrier

A median barrier is "*a longitudinal barrier used to prevent an errant vehicle from crossing the highway median* (8)." The AASHTO *Roadside Design Guide* provides performance requirements, placement guidelines, and structural and safety characteristics of different median barrier systems (1).

Rural multilane highways

Installing any type of median barrier on rural multilane highways reduces fatal-and-injury crashes of all types, as shown in Table 13-23 (8).

The base condition of the CMFs (i.e., the condition in which the CMF = 1.00) is the absence of a median barrier.

Table 13-23. Potential Crash Effects of Installing a Median Barrier (8)

Treatment	Setting (Road Type)	Traffic Volume	Crash Type (Severity)	CMF	Std. Error
Install any type of median barrier	Unspecified (Multilane divided highways)	AADT of 20,000 to 60,000	All types (Fatal)	**0.57?**	**0.1**
			All types (Injury)	**0.70?**	**0.06**
			All types (All severities)	**1.24?**	**0.03**
Install steel median barrier			All types (Injury)	**0.65**	**0.08**
Install cable median barrier				**0.71**	**0.1**
Base Condition: Absence of a median barrier.					

NOTE: Based on U.S. studies: Billion 1956; Moskowitz and Schaefer 1960; Beaton, Field and Moskowitz 1962; Billion and Parsons 1962; Billion, Taragin and Cross 1962; Sacks 1965; Johnson 1966; Williston 1969; Galati 1970; Tye 1975; Ricker, Banks, Brenner, Brown and Hall 1977; Hunter, Steward and Council 1993; Sposito and Johnston 1999; Hancock and Ray 2000; Hunter et al 2001; and international studies: Moore and Jehu 1968; Good and Joubert 1971; Andersen 1977; Johnson 1980; Statens vagverk 1980; Martin et al 1998; Nilsson and Ljungblad 2000.
Bold text is used for the most reliable CMFs. These CMFs have a standard error of 0.1 or less.
? Treatment results in a decrease in fatal-and-injury crashes and an increase in crashes of all severities. See Part D—Introduction and Applications Guide.
Width of the median where the barrier was installed and the use of barrier warrants are unspecified.

13.5.2.5. Install Crash Cushions at Fixed Roadside Features

Rural two-lane roads, rural multilane highways, freeways, expressways, and urban and suburban arterials

The crash effects of installing crash cushions at fixed roadside features are shown in Table 13-24 (8). The crash effects for fatal and non-injury crashes with fixed objects are also shown in Table 13-24 (12). The base condition of the CMFs (i.e., the condition in which the CMF = 1.00) is the absence of crash cushions.

Table 13-39. Potential Crash Effects of Placing Edgeline and Centerline Markings (8)

Treatment	Setting (Road Type)	Traffic Volume	Crash Type (Severity)	CMF	Std. Error
Place edgeline and centerline markings	Rural (Two-lane/ Multilane undivided)	Unspecified	All types (Injury)	**0.76**	**0.1**
Base Condition: Absence of markings.					

NOTE: Based on U.S. study: Tamburri, Hammer, Glennon and Lew, 1968. Study does not report if the roadway segments meet MUTCD guidelines for applying edgeline and centerline markings.
Bold text is used for the most reliable CMFs. These CMFs have a standard error of 0.1 or less.

13.8.2.6. Install Edgelines, Centerlines, and PMDs

Edgeline markings, centerline markings, and PMDs are often combined on roadway segments.

Rural two-lane roads, and rural multilane highways

The crash effects of installing edgelines, centerlines, and PMDs where no markings exist are shown in Table 13-40. The base condition of the CMF (i.e., the condition in which the CMF = 1.00) is the absence of markings.

Table 13-40. Potential Crash Effects of Installing Edgelines, Centerlines, and PMDs (8)

Treatment	Setting (Road Type)	Traffic Volume	Crash Type (Severity)	CMF	Std. Error
Install edgelines, centerlines, and PMDs	Urban/Rural (Two-lane/multilane undivided)	Unspecified	All types (Injury)	**0.55**	**0.1**
Base Condition: Absence of markings.					

NOTE: Based on U.S. studies: Tamburri, Hammer, Glennon and Lew 1968, Roth 1970.
Bold text is used for the most reliable CMFs. These CMFs have a standard error of 0.1 or less.

13.8.2.7. Install Snowplowable, Permanent RPMs

Installing snowplowable, permanent RPMs requires consideration of traffic volumes and horizontal curvature (2).

Rural two-lane roads

The crash effects of installing snowplowable, permanent RPMs on low volume (AADT of 0 to 5,000), medium volume (AADT of 5,001 to 15,000), and high volume (AADT of 15,001 to 20,000) roads are shown in Table 13-41 (2).

The varying crash effect by traffic volume is likely due to the lower design standards (e.g., narrower lanes, narrower shoulders, etc.) associated with low-volume roads (2). Providing improved delineation, such as RPMs, may cause drivers to increase their speeds. The varying crash effect by curve radius is likely related to the negative impact of speed increases (2). The base condition of the CMFs (i.e., the condition in which the CMF = 1.00) is the absence of RPMs.

Table 13-54. Summary of Roadway Treatments for Pedestrians and Bicyclists

HSM Section	Treatment	Rural Two-Lane Road	Rural Multilane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
Appendix 13A.9.1.1	Provide a sidewalk or shoulder	N/A	N/A	N/A	N/A	T	—
Appendix 13A.9.1.2	Install raised pedestrian crosswalks	N/A	N/A	N/A	N/A	T	T
Appendix 13A.9.1.3	Install pedestrian-activated flashing yellow beacons with overhead signs	N/A	N/A	N/A	N/A	T	T
Appendix 13A.9.1.4	Install pedestrian-activated flashing yellow beacons with overhead signs and advance pavement markings	N/A	N/A	N/A	N/A	T	T
Appendix 13A.9.1.5	Install overhead electronic signs with pedestrian-activated crosswalk flashing beacons	N/A	N/A	N/A	N/A	T	—
Appendix 13A.9.1.6	Reduce posted speed limit through school zones during school times	T	T	N/A	N/A	T	T
Appendix 13A.9.1.7	Provide pedestrian overpasses and underpasses	—	—	N/A	N/A	T	T
Appendix 13A.9.1.8	Mark crosswalks at uncontrolled locations, intersection or mid-block	—	N/A	N/A	N/A	T	T
Appendix 13A.9.1.9	Use alternative crosswalk markings at mid-block locations	—	N/A	N/A	N/A	T	T
Appendix 13A.9.1.10	Use alternative crosswalk devices at mid-block locations	—	N/A	N/A	N/A	T	T
Appendix 13A.9.1.11	Provide a raised median or refuge island at marked and unmarked crosswalks	N/A	N/A	N/A	N/A	T	T
Appendix 13A.9.1.12	Provide a raised or flush median or center two-way left-turn lane at marked and unmarked crosswalks	N/A	N/A	N/A	N/A	T	T
Appendix 13A.9.1.13	Install pedestrian refuge islands or split pedestrian crossovers	N/A	N/A	N/A	N/A	T	T
Appendix 13A.9.1.14	Widen median	N/A	—	N/A	N/A	T	T
Appendix 13A.9.1.15	Provide dedicated bicycle lanes (BLs)	N/A	N/A	N/A	N/A	T	—
Appendix 13A.9.1.16	Provide wide curb lanes (WCLs)	N/A	N/A	N/A	N/A	T	—
Appendix 13A.9.1.17	Provide shared bus/bicycle lanes	N/A	N/A	N/A	N/A	T	—
Appendix 13A.9.1.18	Re-stripe roadway to provide bicycle lane	N/A	N/A	N/A	N/A	T	—
Appendix 13A.9.1.19	Pave highway shoulders for bicycles	T	T	N/A	N/A	N/A	—
Appendix 13A.9.1.20	Provide separate bicycle facilities	N/A	N/A	N/A	N/A	T	—

NOTE: T = Indicates that a CMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix 13A.
N/A = Indicates that the treatment is not applicable to the corresponding setting.

$$CMF = \frac{0.322 + DD \times [0.05 - 0.005 \times \ln(AADT)]}{0.322 + 5 \times [0.05 - 0.005 \times \ln(AADT)]} \quad (13\text{-}7)$$

Where:

$AADT$ = average annual daily traffic volume of the roadway being evaluated; and

DD = access point density measured in driveways per mile.

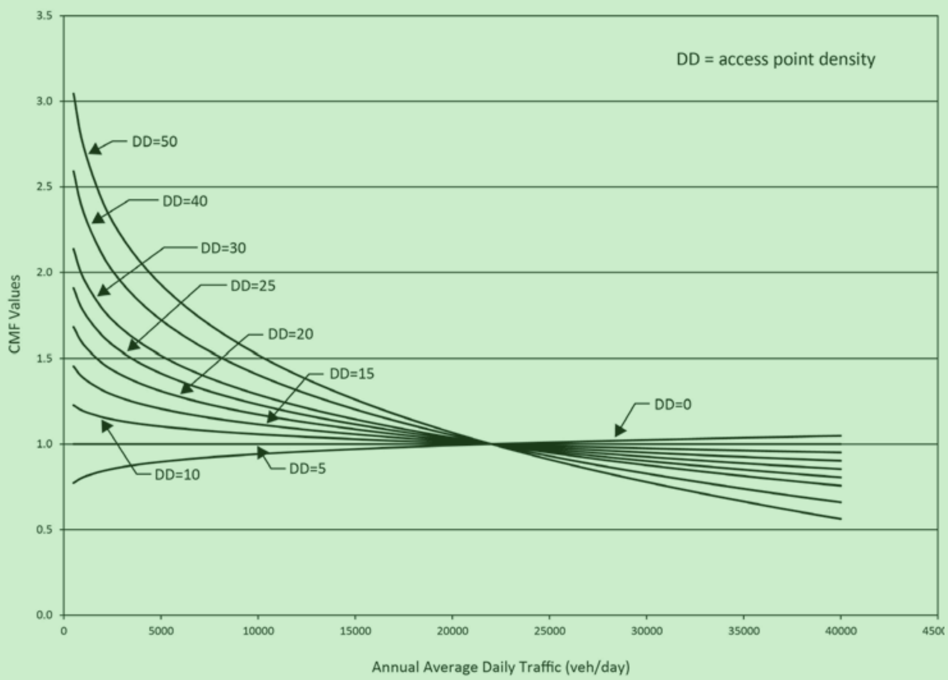

Figure 13-11. Potential Crash Effects of Access Point Density on Rural Two-Lane Roads

Urban and Suburban Arterials

The crash effects of decreasing access point density on urban and suburban arterials are shown in Table 13-58 (8).

The base condition of the CMFs (i.e., the condition in which the CMF = 1.00) is the initial driveway density prior to the implementation of the treatment as presented in Table 13-58.

Table 13-58. Potential Crash Effects of Reducing Access Point Density (8)

Treatment	Setting (Road Type)	Traffic Volume	Crash Type (Severity)	CMF	Std. Error
Reduce driveways from 48 to 26–48 per mile	Urban and suburban (Arterial)	Unspecified	All types (Injury)	**0.71**	**0.04**
Reduce driveways from 26–48 to 10–24 per mile				**0.69**	**0.02**
Reduce driveways from 10–24 to less than 10 per mile				**0.75**	**0.03**
Base Condition: Initial driveway density per mile based on values in this table (48, 26–48, and 10–24 per mile).					

NOTE: Based on international studies: Jensen 1968; Grimsgaard 1976; Hvoslef 1977; Amundsen 1979; Grimsgaard 1979; Hovd 1979; Muskaug 1985.
Bold text is used for the most reliable CMFs. These CMFs have a standard error of 0.1 or less.

Table 14-3. Potential Crash Effects of Converting a Signalized Intersection into a Modern Roundabout (29)

Treatment	Setting (Intersection Type)	Traffic Volume	Crash Type (Severity)	CMF	Std. Error
Convert signalized intersection to modern roundabout	Urban (One or two lanes)	Unspecified	All types (All severities)	**0.99***	**0.1**
			All types (Injury)	**0.40**	**0.1**
	Suburban (Two lanes)		All types (All severities)	**0.33**	**0.05**
	All settings (One or two lanes)		All types (All severities)	**0.52**	**0.06**
			All types (Injury)	**0.22**	**0.07**
Base Condition: Signalized intersection.					

NOTE: **Bold** text is used for the most reliable CMFs. These CMFs have a standard error of 0.1 or less.
*Observed variability suggests that this treatment could result in an increase, decrease, or no change in crashes. See Part D—Introduction and Applications Guidance.
The study from which this information was obtained does not contain information related to the posted or observed speeds at or on approach to the intersections that were converted to a modern roundabout.

If the setting is known, it is recommended that the corresponding urban/suburban CMF be used rather than the CMF for "All settings."

Information regarding pedestrians and bicyclists at modern roundabouts is contained in Appendix 14A.

14.4.2.3. Convert a Stop-Controlled Intersection to a Modern Roundabout

Urban, suburban, and rural stop-controlled intersections

Table 14-4 summarizes the crash effects related to:

- Converting an intersection with minor-road stop control into a modern roundabout;
- Converting a rural intersection with minor-road stop control into a one-lane modern roundabout;
- Converting an urban intersection with minor-road stop control into a one-lane modern roundabout;
- Converting an urban intersection with minor-road stop control into a two-lane modern roundabout;
- Converting a suburban intersection with minor-road stop control into a one-lane or two-lane modern roundabout; and
- Converting an all-way, stop-controlled intersection in any setting into a modern roundabout.

The predictive method for urban and suburban arterials in Chapter 12 includes a procedure for roundabouts at intersections that previously had minor-road stop control. This procedure is based on the CMF for installing modern roundabouts in all settings presented in Table 14-4.

The base condition for the CMFs shown in Table 14-4 (i.e., the condition in which the CMF = 1.00) is a stop-controlled intersection.

Change in Expected Average Crash Frequency:
15.0 – 13.0 = 2.0 crashes/year reduction

7) Discussion: This example shows that expected average crash frequency may potentially be reduced by 2.0 crashes/year with the skew angle variation from 45 to 10 degrees. A standard error was not available for this CMF, therefore a confidence interval for the reduction cannot be calculated.

Intersections on rural multilane highways

The crash effect of skew angle for three-leg intersections with minor-road stop control is represented by (20):

$$CMF = \frac{0.016 \times skew}{(0.98 + 0.016 \times skew)} + 1.0 \tag{14-3}$$

This CMF applies to total intersection crashes. The analogous CMF for four-leg intersections with minor-road stop control is (20):

$$CMF = \frac{0.053 \times skew}{(1.43 + 0.053 \times skew)} + 1.0 \tag{14-4}$$

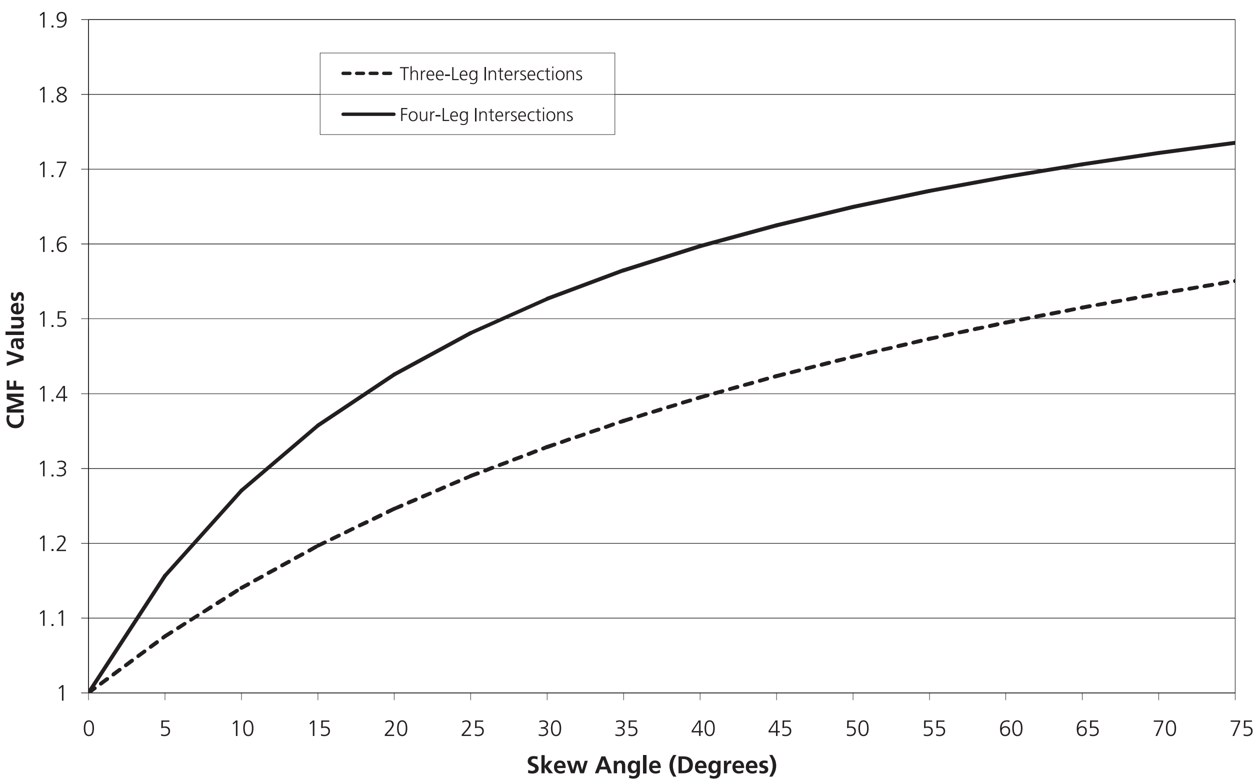

Figure 14-7. Potential Crash Effects of Skew Angle of Three- and Four-Leg Intersections with Minor-Road Stop Control on Rural Multilane Highways

Equivalent CMFs for the crash effect of intersection skew on fatal-and-injury crashes (excluding possible-injury crashes, also known as C-injury crashes) for three-leg intersections with minor-road stop control are presented as Equations 14-5 and 14-6 (20):

$$CMF_{kab} = \frac{0.017 \times skew}{(0.52 + 0.017 \times skew)} + 1.0 \quad (14\text{-}5)$$

Where:

CMF_{kab} = CMF for fatal-and-injury crashes (excluding possible-injury crashes, also known as C-injury crashes).

For four-leg intersections with minor-road stop control (20):

$$CMF_{kab} = \frac{0.048 \times skew}{(0.72 + 0.048 \times skew)} + 1.0 \quad (14\text{-}6)$$

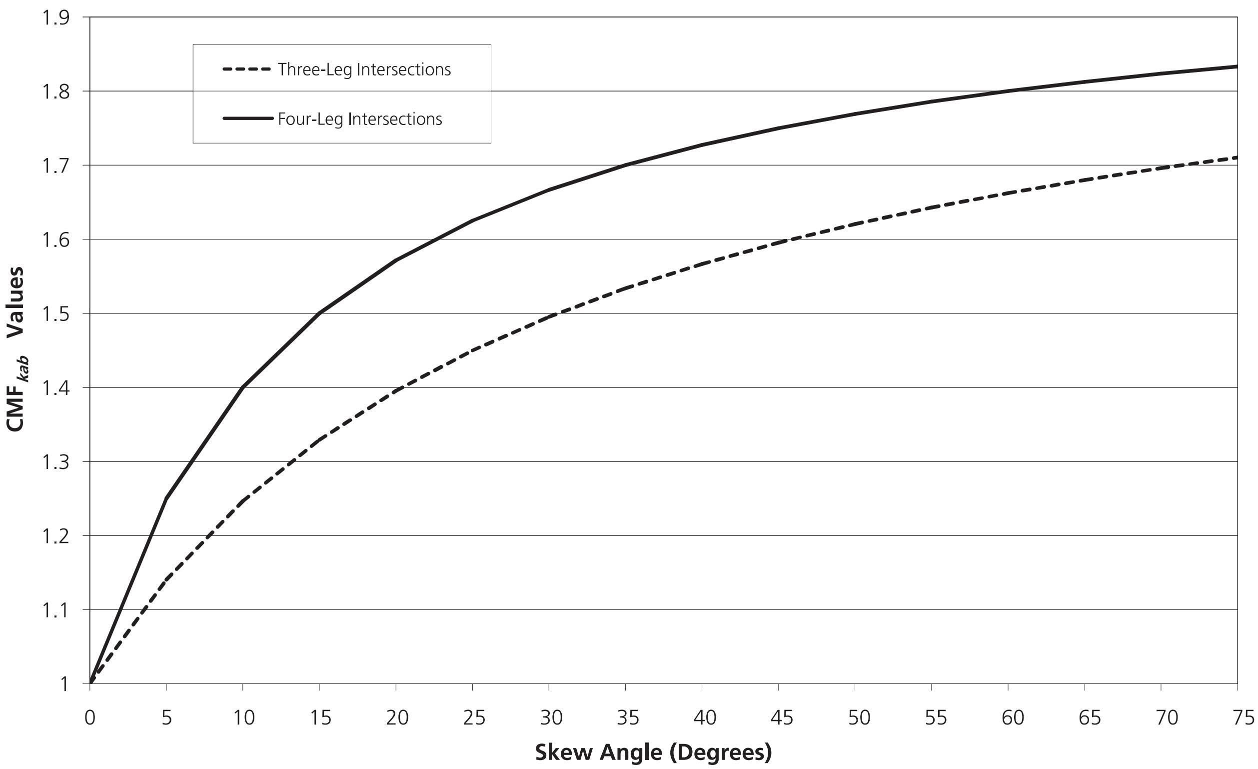

Figure 14-8. Potential Crash Effects of Skew Angle on Fatal-and-Injury Crashes for Three- and Four-Leg Intersections with Minor-Road Stop Control

The CMFs presented in Equations 14-3 through 14-6 are used in the predictive method for rural multilane highways in Chapter 11 to represent the effect of intersection skew at intersections with minor-road stop control. The variability of these CMFs is unknown.

14.6.2.2. Provide a Left-Turn Lane on One or More Approaches to Three-Leg Intersections
Urban and rural three-leg, minor-road, stop-controlled intersections, and urban and rural three-leg signalized intersections

By removing left-turning vehicles from the through-traffic stream, conflicts with through vehicles can be reduced or even eliminated depending on the signal timing and phasing scheme. Providing a left-turn lane allows drivers to wait in the turn lane until a gap in the opposing traffic allows them to turn safely. The left-turn lane helps to reduce conflicts with opposing through traffic (3).

14.7.2.8. Install Red-Light Cameras at Intersections

Various Intelligent Transportation System (ITS) treatments are available for at-grade intersections. Treatments include signal coordination, red-light hold systems, queue detection systems, automated enforcement, and red-light cameras. At the time of this edition of the HSM, red-light cameras were the only treatment for which the crash effects were better understood. This section discusses the effects on crash frequency of installing red-light cameras.

Red-light cameras are positioned along the approaches to intersections with traffic signals to detect and record the occurrence of red-light violations. Installing red-light cameras and the associated enforcement program is generally accompanied by signage and public information programs.

Urban signalized intersections

The crash effects of installing red-light cameras at urban signalized intersections are shown in Table 14-28. The base condition for the CMFs shown in Table 14-28 (i.e., the condition in which the CMF = 1.00) is a signalized intersection without red-light cameras.

Table 14-28. Potential Crash Effects of Installing Red-Light Cameras at Intersections (23,30)

Treatment	Setting (Intersection Type)	Traffic Volume	Crash Type (Severity)	CMF	Std. Error
Install red-light cameras	Urban (Unspecified)	Unspecified	Right-angle and left-turn opposite direction (All severities) (23,30)	**0.74?+**	**0.03**
			Right-angle and left-turn opposite direction (Injury) (23)	**0.84?**	**0.07**
			Rear-end (All severities) (23,30)	**1.18?+**	**0.03**
			Rear-end (Injury) (23)	**1.24?**	**0.1**
Base Condition: A signalized intersection without red-light cameras.					

NOTE: **Bold** text is used for the most reliable CMFs. These CMFs have a standard error of 0.1 or less.
vpd = vehicles per day
+ Combined CMF, see Part D—Introduction and Applications Guidance.
? Treatment results in a decrease in right-angle crashes and an increase in rear-end crashes. See Chapter 3.

It is possible that installing red-light cameras at intersections will result either in a positive spillover effect or in crash migration at nearby intersections or throughout a jurisdiction. A positive spillover effect is the reduction of crashes at adjacent intersections without red-light cameras due to drivers' sensitivity to the possibility of a red-light camera being present. Crash migration is a reduction in crash occurrence at the intersections with red-light cameras and an increase in crashes at adjacent intersections without red-light cameras as travel patterns shift to avoid red-light camera locations. However, the existence and/or magnitude of the crash effects are not certain at this time.

14.8. CONCLUSION

The treatments discussed in this chapter focus on the crash effects of characteristics, design elements, traffic control elements, and operational elements related to intersections. The information presented is the CMFs known to a degree of statistical stability and reliability for inclusion in this edition of the HSM. Additional qualitative information regarding potential intersection treatments is contained in Appendix 14A.

The remaining chapters in Part D present treatments related to other site types such as roadway segments and interchanges. The material in this chapter can be used in conjunction with activities in Chapter 6—Select Countermeasures and Chapter 7—Economic Appraisal. Some Part D CMFs are included in Part C for use in the predictive method. Other Part D CMFs are not presented in Part C but can be used in the methods to estimate change in crash frequency described in Section C.7.

14A.5.1.8. Provide Leading Pedestrian Interval Signal Timing Pattern

A leading pedestrian interval (LPI) is a pre-timed allocation to allow pedestrians to begin crossing the street in advance of the next cycle of vehicle movements. For example, pedestrians crossing the western leg of an intersection are traditionally permitted to cross during the north-south vehicle green phase. Implementing an LPI would provide pedestrians crossing the western leg of the intersection a given amount of time to start crossing the western leg after the east-west vehicle movements and before the north-south vehicle movements. The LPI provides pedestrians an opportunity to begin crossing without concern for turning vehicles (assuming right-on-red is prohibited).

Providing a three-second LPI at signalized intersections with pedestrian signal heads and a one-second, all-red interval appears to reduce conflicts between pedestrians and turning vehicles (40). In addition, a three-second LPI appears to reduce the incidence of pedestrians yielding the right-of-way to turning vehicles, making it easier for pedestrians to cross the street by allowing them to occupy the crosswalk before turning vehicles are permitted to enter the intersection (40).

14A.5.1.9. Provide Actuated Control

The choice between actuated or pre-timed operations is influenced by the practices and standards of the jurisdiction. Intersection-specific characteristics such as traffic flows and intersection design also influence the use of actuated or pre-timed phases.

For the same traffic flow conditions at an actuated signal and pre-timed signal, actuated control appears to reduce some types of crashes compared with pre-timed traffic signals (7). However, the magnitude of the crash effect is not certain at this time.

14A.5.1.10. Operate Signals in “Night-Flash” Mode

Night-flash operation or mode is the use of flashing signals during low-volume periods to minimize delay at a signalized intersection.

Research indicates that replacing night-flash with regular phasing operation may reduce nighttime and nighttime right-angle crashes (19). However, the results are not sufficiently conclusive to determine a CMF for this edition of the HSM.

The crash effect of providing “night-flash” operations appears to be related to the number of approaches to the intersection (8).

14A.5.1.11. Provide Advance Static Warning Signs and Beacons

Traffic signs are typically classified into three categories: regulatory signs, warning signs, and guide signs. As defined in the *Manual on Uniform Traffic Control Devices* (MUTCD) (14), regulatory signs provide notice of traffic laws or regulations, warning signs give notice of a situation that might not be readily apparent, and guide signs show route designations, destinations, directions, distances, services, points of interest, and other geographical, recreational, or cultural information. The MUTCD provides standards and guidance for signage within the right-of-way of all types of highways open to public travel. Many agencies supplement the MUTCD with their own guidelines and standards. This section discusses the crash effects of providing advance static warning signs with beacons.

Providing advance static warning signs with beacons prior to an intersection appears to reduce crashes (9). This treatment may have a larger crash effect when drivers do not expect an intersection or have limited visibility to the intersection ahead (5). However, the magnitude of the crash effect is not certain at this time.

14A.5.1.12. Provide Advance Warning Flashers and Warning Beacons

An advance warning flasher (AWF) is a traffic control device that provides drivers with advance information on the status of a downstream traffic signal. AWFs may be responsive (i.e., linked to the signal timing mechanism) or continuous. Continuous AWFs are also called warning beacons.

The crash effects of responsive AWFs appear to be related to entering traffic flows from minor- and major-road approaches (38).

15.4. CRASH EFFECTS OF INTERCHANGE DESIGN ELEMENTS

15.4.1. Background and Availability of CMFs

Table 15-1 lists common treatments related to interchange design and the CMFs available in this edition of the HSM. Table 15-1 also contains the section number where each CMF can be found.

Table 15-1. Treatments Related to Interchange Design

HSM Section	Treatment	Trumpet	One Quadrant	Diamond	Single Point Urban	Partial Cloverleaf	Full Cloverleaf	Directional
15.4.2.1	Convert intersection to grade-separated interchange	✓	✓	✓	✓	✓	✓	✓
15.4.2.2	Design interchange with crossroad above freeway	✓	—	✓	—	✓	✓	—
15.4.2.3	Modify speed change lane design	✓	✓	✓	✓	✓	✓	✓
15.4.2.4	Modify two-lane-change merge/diverge area to one-lane-change	✓	✓	✓	✓	✓	✓	✓
Appendix 15A.2.2.1	Redesign interchange to modify interchange configuration	T	T	T	T	T	T	T
Appendix 15A.2.2.2	Modify interchange spacing	T	T	T	T	T	T	T
Appendix 15A.2.2.3	Provide right-hand exit and entrance ramps	T	T	T	T	T	T	T
Appendix 15A.2.2.4	Increase horizontal curve radius of ramp roadway	T	T	T	T	T	T	T
Appendix 15A.2.2.5	Increase lane width of ramp roadway	T	T	T	T	T	T	T
Appendix 15A.2.2.6	Increase length of weaving areas between adjacent entrance and exit ramps	T	T	T	T	T	T	T
Appendix 15A.2.2.7	Redesign interchange to provide collector-distributor roads	T	T	T	T	T	T	T
Appendix 15A.2.2.8	Provide bicycle facilities at interchange ramp terminals	T	T	T	T	T	T	T

NOTE: ✓ = Indicates that a CMF is available for this treatment.
T = Indicates that a CMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix 15A.
— = Indicates that a CMF is not available and a crash trend is not known.

15.4.2.3. Modify Speed Change Lane Design

A speed change lane typically connects two facilities with differing speed limits. Speed change lanes include acceleration and deceleration lanes at on-ramps and off-ramps respectively. Speed change lanes include several design elements, such as lane width, shoulder width, length, and taper design.

CMF functions for acceleration lane length are incorporated in the FHWA Interchange Safety Analysis Tool (ISAT) software as follows (2,6):

For total crashes (all severity levels combined):

$$CMF = 1.296 \times e^{(-2.59 \times L_{accel})} \tag{15-1}$$

For fatal-and-injury crashes:

$$CMF = 1.576 \times e^{(-4.55 \times L_{accel})} \tag{15-2}$$

Where:

L_{accel} = length of acceleration lane (mi).

L_{accel} is measured from the nose of the gore area to the end of the lane drop taper. The base condition for the CMFs in Equations 15-1 and 15-2 is a 0.1-mi- (528-ft-) long acceleration lane. The variability of these CMFs is unknown.

If an acceleration lane with an existing length other than 0.1 mi (528 ft) is lengthened, a CMF for that change in length can be computed as a ratio of two values computed with Equations 15-1 and 15-2. For example, if an acceleration lane with a length of 0.12 mi (634 ft) were lengthened to 0.20 mi (1,056 ft), the applicable CMF for total crashes would be the ratio of the CMF determined with Equation 15-1 for the existing length of 0.20 mi (1,056 ft) to the CMF determined with Equation 15-1 for the proposed length of 0.12 mi (634 ft), this calculation is illustrated in Equation 15-3.

$$CMF = \frac{1.576 \times e^{(-4.55 \times 0.20)}}{1.576 \times e^{(-4.55 \times 0.12)}} = 0.69 \tag{15-3}$$

The crash effects and standard error associated with increasing the length of a deceleration lane that is currently 690 ft or less in length by about 100 ft is shown in Table 15-4 (4).

The base condition of the CMFs in Table 15-4 (i.e., the condition in which the CMF = 1.00) is maintaining the existing deceleration lane length of less than 690 ft. The CMF in Table 15-4 may be extrapolated in proportion to the change in lane length for increases in length of less than or more than 100 ft as long as the resulting deceleration lane length does not exceed 790 ft.

Table 15-4. Potential Crash Effects of Extending Deceleration Lanes (4)

Treatment	Setting (Interchange Type)	Traffic Volume	Crash Type (Severity)	CMF	Std. Error
Extend deceleration lane by approx. 100 ft	Unspecified (Unspecified)	Unspecified	All types (All severities)	**0.93***	**0.06**
Base Condition: Maintain existing deceleration lane that is less than 690 ft in length.					

NOTE: **Bold** text is used for the more statistically reliable CMFs. These CMFs have a standard error of 0.1 or less.
* Observed variability suggests that this treatment could result in an increase, decrease, or no change in crashes. See Part D—Introduction and Applications Guidance.

areas are inherent in the design of full cloverleaf interchanges but can occur in or between other interchange types. Short weaving areas between adjacent entrance and exit ramps have been found to be associated with increased crash frequencies. Research indicates that providing longer weaving areas will reduce crashes (1). However, the available research is not sufficient to develop a quantitative CMF.

15A.2.2.7. Redesign Interchange to Provide Collector-Distributor Roads

Crashes associated with weaving areas within an interchange or between adjacent interchanges can be reduced by redesigning the interchange(s) to provide collector-distributor roads. This design moves weaving from the mainline freeway to an auxiliary roadway, typically reducing both the volumes and the traffic speeds in the weaving area. The addition of collector-distributor roads has been shown to reduce crashes (7,9). However, the available research is not sufficient to develop a quantitative CMF.

15A.2.2.8. Provide Bicycle Facilities at Interchange Ramp Terminals

Continuity of bicyclist facilities can be provided at interchange ramp terminals. Bicyclists are considered vulnerable road users as they are more susceptible to injury when involved in a traffic crash than vehicle occupants. Vehicle occupants are usually protected by the vehicle.

Bicyclists must sometimes cross interchange ramps at uncontrolled locations. Encouraging bicyclists to cross interchange ramps at right angles appears to increase driver sight distance and reduce the bicyclists' risk of a crash (5).

15A.3. TREATMENTS WITH UNKNOWN CRASH EFFECTS

15A.3.1. Treatments Related to Interchange Design

Merge/Diverge Areas

- Modify merge/diverge design (e.g., parallel versus taper, left-hand versus right-hand)
- Modify roadside design or elements at merge/diverge areas
- Modify horizontal and vertical alignment of the merge or diverge area
- Modify gore area design

Ramp Roadways

- Increase shoulder width of ramp roadway
- Modify shoulder type of ramp roadway
- Provide additional lanes on the ramp
- Modify roadside design or elements on ramp roadways
- Modify vertical alignment of the ramp roadway
- Modify superelevation of ramp roadway
- Provide two-way ramps
- Provide directional ramps
- Modify ramp design speed
- Provide high-occupancy vehicle lanes on ramp roadways
- Modify ramp type or configuration

Ramp Terminals

- Modify ramp terminal intersection type
- Modify ramp terminal approach cross-section
- Modify ramp terminal roadside elements
- Modify ramp terminal alignment elements
- Provide direct connection or access to commercial or private sites from ramp terminal
- Provide physically channelized right-turn lanes

Bicyclists and Pedestrians

- Provide pedestrian and/or bicyclist traffic control devices at ramp terminals
- Provide refuge islands
- Provide pedestrian facilities on ramp terminals
- Develop policies related to pedestrian and bicyclist activity at interchanges

15A.3.2. Treatments Related to Interchange Traffic Control and Operational Elements

Traffic Control at Ramp Terminals

- Provide traffic signals at ramp terminal intersection
- Provide stop-control or yield-control signs at ramp terminal intersections

15A.4. APPENDIX REFERENCES

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Table 16-4. Treatments Related to Work Zone Design Elements

HSM Section	Treatment	Rural Two-Lane Road	Rural Multilane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
16.4.2.1	Modify work zone duration and length	—	—	✓	—	—	—
Appendix 16A.3.2	Use crossover closure or single lane closure	—	T	T	T	—	—
Appendix 16A.3.3	Use Indiana Lane Merge System (ILMS)	—	—	T	—	—	—

NOTE: ✓ = Indicates that a CMF is available for the treatment.
T = Indicates that a CMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix 16A.
— = Indicates that a CMF is not available and a crash trend is not known.

16.4.2. Work Zone Design Treatments with CMFs

16.4.2.1. Modify Work Zone Duration and Length

Freeways

Work zone design elements include the duration in the number of days and the length in miles. Equation 16-1 and Figure 16-1 present a CMF for the potential crash effects of modifying the work zone duration. Equation 16-2 and Figure 16-2 present a CMF for the potential crash effects of modifying the work zone length. These CMFs are based on research that considered work zone durations from 16 to 714 days, work zone lengths from 0.5 to 12.2 mi, and freeway AADTs from 4,000 to 237,000 veh/day (8).

The base condition of the CMFs (i.e., the condition in which the CMF = 1.00) is a work zone duration of 16 days and/or work zone length of 0.51 miles. The standard errors of the CMFs below are unknown.

Expected average crash frequency effects of increasing work zone duration (8)

$$CMF_{all} = 1.0 + \frac{(\%\text{ increase in duration} \times 1.11)}{100} \tag{16-1}$$

Where:

CMF_{all} = crash modification factor for all crash types and all severities in the work zone; and

% increase in duration = the percentage change in the duration (days) of the work zone.

islation, and enforcement levels. Road-use culture evolves as individuals influence society and as society influences individuals. Additional information regarding road-use culture can be found in Appendix 17A.

Table 17-4 summarizes treatments related to road-use culture and the corresponding CMFs available. The treatments summarized below encompass engineering, enforcement, and education.

Table 17-4. Road-Use Culture Network Considerations and Treatments

HSM Section	Treatment	Urban	Suburban	Rural
17.5.2.1	Install automated speed enforcement	✓	—	✓
17.5.2.2	Install changeable speed warning signs	✓	✓	✓
Appendix 17A.4.1.1	Deploy mobile patrol vehicles	T	T	T
Appendix 17A.4.1.2	Deploy stationary patrol vehicles	T	T	T
Appendix 17A.4.1.3	Deploy aerial enforcement	T	T	T
Appendix 17A.4.1.4	Deploy radar and laser speed monitoring equipment	T	T	T
Appendix 17A.4.1.5	Install drone radar	T	T	T
Appendix 17A.4.1.6	Modify posted speed limit	T	T	T
Appendix 17A.4.1.7	Conduct enforcement to reduce red-light running	T	T	T
Appendix 17A.4.1.8	Conduct enforcement to reduce impaired driving	T	T	T
Appendix 17A.4.1.9	Conduct enforcement to increase seat belt and helmet use	T	T	T
Appendix 17A.4.1.10	Implement network-wide engineering consistency	T	T	T
Appendix 17A.4.1.11	Conduct public education campaigns	T	T	T
Appendix 17A.4.1.12	Implement young drivers and graduated driver licensing programs	T	T	T

NOTE: ✓ = Indicates that a CMF is available for the treatment.
T = Indicates that a CMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix 17A.
— = Indicates that a CMF is not available and a trend is not known.

17.5.2. Road Use Culture Network Consideration Treatments with CMFs

17.5.2.1. Install Automated Speed Enforcement

Automated enforcement systems use video or photographic identification in conjunction with radar or lasers to detect speeding drivers. The systems automatically record vehicle registrations without needing police officers at the scene.

The crash effects of installing automated speed enforcement in urban or rural areas on all road types are shown in Table 17-5 (1,3,5,7,9,12). The base condition for this CMF (i.e., the condition in which the CMF = 1.00) is the absence of automated speed enforcement.

17A.4.1.6. Modify Posted Speed Limit

Drivers tend to drive at the speed that they find acceptable and safe, despite posted speed limits.

Little or no effect on operating speed has been found for low- and moderate-speed roads where posted speed limits were raised or lowered (20). On high-speed roads such as freeways, "studies in the USA and abroad generally show an increase in speeds when speed limits are raised (20)."

The net crash effect of speed limits and changes in speed limits across the transportation network is not fully known. More information is needed to understand how drivers respond to speed limits and how driver behavior can be modified. This information would help to improve how speed limits are set and would help to maximize the results of speed enforcement efforts.

17A.4.1.7. Conduct Enforcement to Reduce Red-Light Running

Automated enforcement for red-light running, combined with appropriate enabling legislation, potentially reduces crashes.

17A.4.1.8. Conduct Enforcement to Reduce Impaired Driving

Although alcohol and drugs have a major effect on driver error, and although driving under the influence (DUI) of alcohol or other drugs is widely regarded as a major problem, attitudes towards drinking and driving are not fully understood.

Behavioral controls appear to provide the best results for reducing drunk driving among people with multiple DUI offenses (8). Behavioral controls include internal behavior controls such as moral beliefs concerning alcohol-impaired driving, and external behavioral controls such as the offenders' perceptions of crashes and criminal punishment. Social controls or peer group pressure appear to be less effective.

Many approaches have been tried to reduce DUI, including:

1. Instituting classes for juvenile DUI offenders;

2. Providing alcohol abuse treatment as an alternative to license suspensions;

3. Lowering the legal blood alcohol limit to 0.05;

4. Introducing random breath testing;

5. Training bar staff;

6. Setting up highly publicized sobriety checkpoints;

7. Implementing underage drinking controls;

8. Limiting alcohol availability;

9. Using media advocacy; and

10. Punishing offenders, including ignition interlock devices or impounding vehicles for repeat offenders.

The first five approaches do not result in a clear pattern of driver response. Some drivers are frequent violators and appear to need special attention and policies (16).

As an example of a more severe approach, DUI laws introduced in California in 1990 included a pre-conviction license suspension on arrested DUI offenders. The approach was " . . . highly effective in reducing subsequent crashes and recidivism among DUI offenders (18)."

On the other hand, some evidence shows a multipronged approach may be a more effective choice. "Drinking and driving prevention seems to be most successful when it engages a broad variety of programs and interventions (23)." Such a program in Salinas, California ". . . succeeded not only in mobilizing the community, but also in reducing traffic injuries and impaired driving over a sustained period of time. Traffic crashes, injuries, and drinking and driving rates all decreased as a result of the project (23)." Programs that concentrated only on sobriety checkpoints appear to reduce crash frequency and increase DUI arrests over the short-term but are not successful over the long term (23).

These DUI approaches suggest that road-use culture can be modified but that change requires concentrated legislation and enforcement efforts, as well as appropriate community programs, to achieve long-term and sustainable results.

17A.4.1.9. Conduct Enforcement to Increase Seat Belt and Helmet Use

The effectiveness of enforcing seat belt and helmet use is directly related to whether or not the laws are primary or secondary laws. A primary seat belt law allows law enforcement officials to ticket anyone not wearing a seat belt. A secondary seat belt law means that a police officer can only write a ticket for a seat belt violation if the driver is also cited for some other violation. If a seat belt law is secondary, not wearing a seat belt is still against the law; however, enforcement of the law is not as effective.

Adopting primary laws is likely to increase seat belt and helmet use and to modify road-use culture. Primary enforcement may also lead to an increase in seat belt and helmet use.

A change from secondary to primary seat belt use laws has been shown to increase seat belt usage and to decrease driver fatalities (10). Most jurisdictions have supported a change in law with enforcement campaigns. It appears that people are more likely to wear seat belts after legislation (22). "States in which motorists can be stopped solely for belt nonuse had a combined use rate of 85 percent in 2006, compared to 74 percent in other States (7)."

Similarly, universal helmet requirements for motorcyclists increase helmet use. In June 2006, 68 percent of motorcyclists wore helmets that complied with federal safety regulations in states with universal helmet laws, compared with 37 percent in states without a universal helmet law (6).

17A.4.1.10. Implement Network-Wide Engineering Consistency

Network-wide engineering consistency refers to the degree to which a jurisdiction implements transportation engineering solutions using consistent principles and criteria to design transportation infrastructure and to control traffic. Consistently and uniformly applying regulatory, warning, and informational signs is one example. Another example is applying consistent and uniform pavement markings.

The consistency of engineering measures at individual locations and across a jurisdiction's transportation network is likely to affect the driving habits and road-use culture of local users. Road users come to expect certain procedures and to act accordingly. Examples include all-red phases at traffic signals, right-turn-on-red, the use of left-turn arrows or flashing lights at traffic signals, and policies regarding yielding to other vehicles and non-motorized travelers at intersections and roundabouts.

When procedures are not consistent across the jurisdiction, safety may deteriorate. This effect is shown when drivers traveling in a foreign country encounter different rules of the road.

17A.4.1.11. Conduct Public Education Campaigns

Public education campaigns inform road users of new traffic control devices, general rules of the road, and similar topics.

Enforcement efforts can include public information, warnings, or educational campaigns. Such campaigns " . . . contribute significantly to the effectiveness of the technology . . . " used in enforcement, " . . . result in safer driving habits . . . ", and can improve the image of police enforcement activities (20). Extensive pedestrian safety education programs directed at children in elementary schools and those ages 4 to 7 appear to reduce child pedestrian crashes (4).

It is also recognized that not all public information and education (PI&E) programs are effective. A review of some PI&E programs found that the only programs that resulted in a substantial reduction in speed, speeding, crashes, or crash severity were those that were integrated with a law enforcement program (20). "General assessment of public information programs has shown [PI&E programs] to have limited effect on actual behavior except when they are paired with enforcement (14)."

Program effectiveness generally depends on the use of multimedia, careful planning, and professional production. The impact, however, is difficult to measure and extremely difficult to separate from the effects of a campaign's enforcement component (14).

17A.4.1.12. Implement Young Driver and Graduated Driver Licensing Programs

Graduated driver licensing (GDL) programs developed for novice drivers have been implemented in many jurisdictions. GDL programs typically include restrictions such as zero blood alcohol, not driving on high-speed highways, not driving at night, and limitations on the number and age of passengers. The restrictions are designed to encourage new drivers to gain experience under conditions that minimize exposure to risk and to ensure drivers are exposed to more demanding driving situations only when they have enough experience (13). The concern is new drivers are at risk while getting the experience they need.

Novice drivers are three times more likely to be involved in a fatal traffic crash than other drivers (1,24). Evidence also indicates that the most dangerous times and situations for drivers aged 16 to 20 years are (1):

- At night
- On freeways
- Driving with passengers

The level of risk for young drivers suggests that novice drivers need a learning period when they are subject to measures that " . . . minimize their exposure, especially in known risky circumstances like nighttime and on freeways (1)."

Although GDL programs and their results vary, it appears that there is a decrease in crash frequency with a GDL program (13). There is also an indication that "increased driving experience is somewhat more important than increased age in reducing crashes among young novice" drivers (13).

17A.5. TREATMENTS WITH UNKNOWN CRASH EFFECTS

No information about the crash effects of the following treatments was available for this edition of the HSM.

17A.5.1. Network Traffic Control and Operational Elements

- Implement network-wide or area-wide turn restrictions

17A.5.2. Road-Use Culture Network Considerations

- Install enforcement notification signs
- Mitigate aggressive driving through engineering
- Implement older driver education and retesting programs

Table 18-20. Coefficients for Lane Change CMF–Freeway Segments

Cross Section (x)	Crash Type (y)	Crash Severity (z)	CMF Variable	CMF Coefficients			
				a	b	c	d
Any cross section (ac)	Multiple vehicle (mv)	Fatal and injury (fi)	$CMF_{7,\,fs,\,ac,\,mv,\,fi}$	0.175	12.56	0.001	–0.272
		Property damage only (pdo)	$CMF_{7,\,fs,\,ac,\,mv,\,pdo}$	0.123	13.46	0.001	–0.283

If the segment is in a Type B weaving section, then the length of the weaving section is an input to the CMF. The variables for weaving section length (i.e., $L_{wev,\,inc,}$ $L_{wev,\,dec}$) in Equation 18-31 and Equation 18-32 are intended to reflect the degree to which the weaving activity is concentrated along the freeway. The sign of the coefficient in these two equations indicates that the lane change CMF value will increase if the segment is in a Type B weaving section. The amount of this increase is inversely related to the length of the weaving section. Guidance for determining if a weaving section is Type B is provided in Section 18.4.

The variables $P_{wevB,\,inc}$ and $P_{wevB,\,dec}$ in Equation 18-31 and Equation 18-32, respectively, are computed as the ratio of the length of the weaving section in the segment to the length of the freeway segment L_{fs}. If the segment is wholly located in the weaving section, then this variable is equal to 1.0.

The X and $AADT$ variables describe the distance to (and volume of) the four nearest ramps to the subject segment. Two of the ramps of interest are on the side of the freeway with travel in the increasing milepost direction. One ramp on this side of the freeway is upstream of the segment, and one ramp is downstream of the segment. Similarly, one ramp on the other side of the freeway is upstream of the segment and one ramp is downstream. Only those entrance ramps that contribute volume to the subject segment are of interest. Hence, a downstream entrance ramp is not of interest. For similar reasons, an upstream exit ramp is not of interest.

The lane change CMF is applicable to any segment in the vicinity of one or more ramps. It is equally applicable to segments in a weaving section (regardless of the weaving section type) and segments in a non-weaving section (i.e., segments between an entrance ramp and an exit ramp where both ramps have a speed-change lane). If the weaving section is Type B, then an additional adjustment is made using Equation 18-31 and Equation 18-32. The CMF is applicable to weaving section lengths between 0.10 and 0.85 mi. It is applicable to any value for the distance variable X and to the range of ramp AADTs in Table 19-4.

The two SPFs for predicting speed-change-related crash frequency (i.e., Equation 18-20 and Equation 18-22) are not used when evaluating a weaving section because the ramps that form the weaving section do not have a speed-change lane. As a result, the predicted crash frequency for the set of segments that comprise a weaving section will tend to be smaller than that predicted for a similar set of segments located in a non-weaving section but having entrance and exit ramps. This generalization will always be true for weaving sections that are not Type B. It may or may not hold for the Type B weaving section, depending on the length of the weaving section.

$CMF_{8,\,fs,\,ac,\,sv,\,z}$—Outside Shoulder Width

Two CMFs are used to describe the relationship between average outside shoulder width and predicted crash frequency. The SPFs to which they apply are identified in the following list:

- SPF for fatal-and-injury single-vehicle crashes, specified number of lanes (*fs*, *n*, *sv*, *fi*); and
- SPF for property-damage-only single-vehicle crashes, specified number of lanes (*fs*, *n*, *sv*, *pdo*).

The base condition is a 10-ft outside shoulder width. The CMFs are described using the following equation:

$$CMF_{8,\,fs,\,ac,sv,z} = \left(1.0 - \sum_{i=1}^{m} P_{c,i}\right) \times \exp\left(a \times [W_s - 10]\right) + \left(\sum_{i=1}^{m} P_{c,i}\right) \times \exp\left(b \times [W_s - 10]\right) \qquad (18\text{-}35)$$

Where:

$CMF_{8,\,fs,\,ac,\,sv,\,z}$ = crash modification factor for outside shoulder width in a freeway segment with any cross section *ac*, single-vehicle crashes *sv*, and severity *z*; and

W_s = paved outside shoulder width (ft).

The coefficients for Equation 18-35 are provided in Table 18-21. The variable $P_{c,i}$ is computed as the ratio of the length of curve *i* in the segment to the effective length of the freeway segment L^*. The CMF is applicable to shoulder widths in the range of 4 to 14 ft. The "length of curve *i* in the segment" is computed as an average of two values: the length of curve *i* between the segment's begin and end mileposts for roadbed 1 and that for roadbed 2, where each value excludes the length of any coincident speed-change lane that may be present and where the value for a given roadbed is zero if that roadbed is not curved.

Table 18-21. Coefficients for Outside Shoulder Width CMF–Freeway Segments

Cross Section (*x*)	Crash Type (*y*)	Crash Severity (*z*)	CMF Variable	CMF Coefficients	
				a	*b*
Any cross section (*ac*)	Single vehicle (*sv*)	Fatal and injury (*fi*)	$CMF_{8,\,fs,\,ac,\,sv,\,fi}$	–0.0647	–0.0897
		Property damage only (*pdo*)	$CMF_{8,\,fs,\,ac,\,sv,\,pdo}$	0.00	–0.0840

$CMF_{9,\,fs,\,ac,\,sv,\,fi}$—Shoulder Rumble Strips

One CMF is used to describe the relationship between shoulder rumble strip presence and predicted crash frequency. The SPF to which it applies is identified in the following list:

- SPF for fatal-and-injury single-vehicle crashes, specified number of lanes (*fs*, *n*, *sv*, *fi*).

The base condition is no shoulder rumble strips present. The CMF is described using the following equation:

$$CMF_{9,\,fs,ac,sv,\,fi} = \left(1.0 - \sum_{i=1}^{m} P_{c,\,i}\right) \times f_{\tan} + \left(\sum_{i=1}^{m} P_{c,\,i}\right) \times 1.0 \qquad (18\text{-}36)$$

$$f_{\tan} = 0.5 \times \left([1.0 - P_{ir}] \times 1.0 + P_{ir} \times 0.811\right) + 0.5 \times \left([1.0 - P_{or}] \times 1.0 + P_{or} \times 0.811\right) \qquad (18\text{-}37)$$

Where:

$CMF_{9,\,fs,\,ac,\,sv,\,fi}$ = crash modification factor for shoulder rumble strips in a freeway segment with any cross section *ac* and fatal-and-injury (*fi*) single-vehicle (*sv*) crashes;

f_{tan} = factor for rumble strip presence on tangent portions of the segment;

P_{ir} = proportion of effective segment length with rumble strips present on the inside shoulders; and

P_{or} = proportion of effective segment length with rumble strips present on the outside shoulders.

The proportion P_{ir} represents the proportion of the effective segment length with rumble strips present on the inside shoulders. It is computed by summing the length of roadway with rumble strips on the inside shoulder (excluding the length of any rumble strips adjacent to speed-change lanes) in *both* travel directions and dividing by twice the effective freeway segment length L^*. The proportion P_{or} represents the proportion of the effective segment length with rumble strips present on the outside shoulders. It is computed by summing the length of roadway with rumble strips on the outside shoulder (excluding the length of any rumble strips adjacent to speed-change lanes) in *both* travel directions and dividing by twice the effective freeway segment length L^*.

The first summation term "∑" in Equation 18-48 applies to short lengths of barrier in the median. It indicates that the ratio of barrier length $L_{ib,\,i}$ to clearance distance (= $W_{off,\,in,\,i} - W_{is}$) should be computed for each individual length of barrier that is found in the median along the segment (e.g., a barrier protecting a sign support). The continuous median barrier is not considered in this summation. Any clearance distance that is less than 0.75 ft should be set to 0.75 ft. Similarly, if the distance "0.5 × (W_m – 2 ×W_{is} – W_{ib})" is less than 0.75 ft, then W_{icb} should be set to 0.75 ft.

For segments or speed-change lanes with a continuous barrier adjacent to one roadbed (i.e., asymmetric median barrier), the following equations should be used to estimate W_{icb} and P_{ib}.

$$W_{icb} = \frac{2 \times L}{\dfrac{L}{W_{near} - W_{is}} + \sum \dfrac{L_{ib,i}}{W_{off,in,i} - W_{is}} + \dfrac{L - \sum L_{ib,i}}{W_m - 2 \times W_{is} - W_{ib} - W_{near}}} \qquad (18\text{-}50)$$

$$P_{fs+sc,ac,at,K} = \frac{\exp(V_K)}{\dfrac{1.0}{C_{sdf,fs+sc}} + \exp(V_K) + \exp(V_A) + \exp(V_B)} \qquad (18\text{-}51)$$

Where:

W_{near} = "near" horizontal clearance from the edge of the traveled way to the continuous median barrier (measure for both travel directions and use the smaller distance) (ft).

Similar to the previous guidance, the first summation term "∑" in Equation 18-50 applies to short lengths of barrier in the median. The ratio of barrier length L_{ib} to the clearance distance (= $W_{off,\,in,\,i} - W_{is}$) should be computed for each individual length of barrier that is found in the median along the segment. The continuous median barrier is not considered in this summation. Any clearance distance that is less than 0.75 ft should be set to 0.75 ft. Similarly, if the distance "W_{near} – W_{is}" or the distance "W_m – 2 ×W_{is} – W_{ib} – W_{near}" is less than 0.75 ft, then W_{icb} should be set to 0.75 ft.

For segments or speed-change lanes with a depressed median and some short sections of barrier in the median (e.g., bridge rail), the following equations should be used to estimate W_{icb} and P_{ib}:

$$W_{icb} = \frac{\sum L_{ib,i}}{\sum \dfrac{L_{ib,i}}{W_{off,in,i} - W_{is}}} \qquad (18\text{-}52)$$

$$P_{ib} = \frac{\sum L_{ib,i}}{2 \times L} \qquad (18\text{-}53)$$

Any clearance distance (= $W_{off,\,in,\,i} - W_{is}$) that is less than 0.75 ft should be set to 0.75 ft. When a freeway segment is being evaluated, the proportion P_{ib} represents the proportion of the effective segment length with barrier present in the median. It is computed by summing the length of roadway with median barrier (excluding the length of any median barrier adjacent to speed-change lanes) in *both* travel directions and dividing by twice the effective freeway segment length L^*.

For segments or speed-change lanes with depressed medians without a continuous barrier or short sections of barrier in the median, the following equation should be used to estimate P_{ib}:

$$P_{ib} = 0.0 \qquad (18\text{-}54)$$

The input data needed for this procedure are identified in Table 19-42. The first three variables listed represent required input data. Default values are provided for the remaining variables.

Table 19-42. Input Data for Ramp Curve Speed Prediction

Variable	Description	Default Value	Applicable Site Type
X_i	Ramp-mile of the point of change from tangent to curve (PC) for curve *i* (mi) [a]	None	All
R_i	Radius of curve *i* (ft) [b]	None	All
$L_{c,i}$	Length of horizontal curve *i* (mi)	None	All
V_{frwy}	Average traffic speed on the freeway during off-peak periods of the typical day (mi/h)	Estimate as equal to the speed limit	All
V_{xroad}	Average speed at the point where the ramp connects to the crossroad (mi/h)	15 – ramps with stop-, yield-, or signal-controlled crossroad ramp terminals 30 – all other ramps at service interchanges	Entrance ramp, exit ramp, connector ramp at service interchange
V_{cdroad}	Average speed on C-D road or connector ramp (measured at the mid-point of the C-D road or ramp) (mi/h)	40	C-D road, connector ramp at system interchange

[a] If the curve is preceded by a spiral transition, then X_i is computed as equal to the average of the TS and SC ramp-mile locations, where TS is the point of change from tangent to spiral and SC is the point of change from spiral to curve.
[b] If the curve has spiral transitions, then R_i is equal to the radius of the central circular portion of the curve.

The curve entry speeds need to be calculated for all curves from milepost 0.0 to the end of the analysis segment. This may include segments of an adjacent ramp that are not included in the current analysis segment. For each curve, record the entry speed, the total length of the curve, and the length of the current analysis segment. Once the procedure on the following pages is completed, return to Equation 19-33. In this equation, the summation term only includes entry speeds and radii that have a length in the current analysis segment. All other curves analyzed should be ignored if they are not part of the current analysis segment.

Entrance Ramp Procedure

This procedure is applicable to entrance ramps and connector ramps at service interchanges that serve motorists traveling from the crossroad to the freeway.

Step 1—Gather Input Data.

The input data needed for this procedure are identified in Table 19-42.

Step 2—Compute Limiting Curve Speed.

The limiting curve speed is computed for each curve on the ramp using the following equation:

$$v_{\max,i} = 3.24 \times \left(32.2 \times R_i\right)^{0.30} \quad (19\text{-}59)$$

where $v_{max,i}$ is the limiting speed for curve *i* (ft/s).

The analysis proceeds in the direction of travel. The first curve encountered is curve 1 (*i* =1). The value of v_{max} is computed for all curves prior to, and including, the curve of interest. The value obtained from Equation 19-59 repre-