



## Errata

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The following changes were made to the document after publication:

Location	Incorrect Values	Corrected Values
Page i, Block 21	63	68
Page 4, SAFETY EFFECTS OF HORIZONTAL ALIGNMENT, Paragraph 1, Sentence 1, Line 2	follows	shown in figure 2
Page 4, SAFETY EFFECTS OF HORIZONTAL ALIGNMENT, Paragraph 1, Sentence 1, Figure 2	$N = [1.55 (LC \square (V) + 0.014(D)(V) - 0.012(S)(V))(0.978)W - 30$	$N = [1.55 (LC \square (V) + 0.014(D)(V) - 0.012(S)(V))(0.978)W - 30$
Page 5, Paragraph 3, Sentence 2	in the following form:	shown in figure 3:
Page 5, Paragraph 6, Sentence 1	in the following form:	as shown in figure 4:
Page 8, Paragraph 1, Sentence 10	$(A = \text{abs}(G1 - G2))$	$[A = \text{abs}(G1 - G2)]$
Page 9, DESCRIPTIVE STATISTICS, Paragraph 1, Sentence 1	(million vehicle miles traveled in the 6-year period (MVMT))	[million vehicle miles traveled in the 6-year period (MVMT)]
Page 14, Paragraph 2, Bullet 2	(this is a standard analysis of variance (ANOVA) using an	[this is a standard analysis of variance (ANOVA) using an

Location	Incorrect Values	Corrected Values
	NB distribution and a log link)	NB distribution and a log link]
Page 16, Paragraph 2, Sentence 1	follows	shown in figure 6 and figure 7
Page 16, Paragraph 4, Bullet 1, Sentence 2	follows	shown in figure 8
Page 17, Horizontal Curves and Tangents on Straight Grades, Paragraph 1, Sentence 1, Bullet 1	(number of roadway segments (N) = 8,095 and total roadway length = 595 mi)	[number of roadway segments (N) = 8,095 and total roadway length = 595 mi]
Page 17, Horizontal Curves and Tangents on Straight Grades, Paragraph 1, Sentence 1	(i.e., number of roadway sections); total roadway length; and minimum, maximum, mean, and median values for specific parameters)	(i.e., number of roadway sections; total roadway length; and minimum, maximum, mean, and median values for specific parameters)
Page 18, Paragraph 1, Sentence 1	follows	shown in figure 10 and figure 11
Page 20, Paragraph 5, Sentence 1	follows	shown in figure 12, figure 13, and figure 14
Page 23, Paragraph 1, Sentence 1	as follows	shown in figure 16 and figure 17
Page 25, Paragraph 2, Sentence 1	follows	shown in figure 18 and figure 19
Page 26, Paragraph 1, Sentence 2	follows	shown in figure 20 and figure 21
Page 28, Paragraph 1, Sentence 1	follows	shown in figure 23 and figure 24
Page 29, Paragraph 5, Sentence 1	follows	shown in figure 25 and figure 26
Page 30, Paragraph 2, Sentence 3	follows	shown in figure 27 and figure 28
Page 32, Paragraph 1, Sentence 1	follows	shown in figure 30 and figure 31
Page 33, Paragraph 4, Sentence 1	follows	shown in figure 32 and figure 33
Page 36, Paragraph 1, Sentence 1	follows	shown in figure 35 and figure 36
Page 37, Paragraph 4, Sentence 1	follows	shown in figure 37 and figure 38
Page 39, HORIZONTAL CURVES AND TANGENTS ON STRAIGHT GRADES, Paragraph 1, Sentence 1	follows	shown in figure 39 and figure 40
Page 42, HORIZONTAL CURVES AND	follows	shown in figure 43 and figure 44

Location	Incorrect Values	Corrected Values
TANGENTS AT TYPE 1 CREST VERTICAL CURVES, Paragraph 1, Sentence 1		
Page 44, HORIZONTAL CURVES AND TANGENTS AT TYPE 1 SAG VERTICAL CURVES, Paragraph 1, Sentence 1	follows	shown in figure 47 and figure 48
Page 46, HORIZONTAL CURVES AND TANGENTS AT TYPE 2 CREST VERTICAL CURVES, Paragraph 1, Sentence 1	follows	shown in figure 51 and figure 52
Page 48, HORIZONTAL CURVES AND TANGENTS AT TYPE 2 SAG VERTICAL CURVES, Paragraph 1, Sentence 1	follows	shown in figure 55 and figure 56
Page 50, CMFs FOR COMBINED CRASH SEVERITY LEVELS, Paragraph 1, Sentence 3	follows	shown in figure 59

# Safety Effects of Horizontal Curve and Grade Combinations on Rural Two-Lane Highways

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Turner-Fairbank Highway Research Center  
6300 Georgetown Pike  
McLean, VA 22101-2296

## FOREWORD

The first edition of the American Association of State Highway and Transportation Officials (AASHTO) *Highway Safety Manual* (HSM) provides crash modification factors (CMFs) for the safety effects of horizontal curvature and percent grade on rural two-lane highways.<sup>(1)</sup> However, the HSM does not provide a method to account for the interactions between these effects. Researchers have always presumed that there are interactions between the safety effects of horizontal and vertical alignment, but this has not been demonstrated for specific alignment combinations in a form useful for safety prediction.

This document is a final report highlighting research undertaken to quantify the safety effects of five specific combinations of horizontal and vertical alignment for rural two-lane highways. The research is based on data from the Federal Highway Administration's Highway Safety Information System (HSIS) for Washington State, including roadway characteristics, horizontal and vertical alignment, traffic volume, and crashes. The outcome is a set of safety prediction models for fatal and injury and property damage only crashes. CMFs representing safety performance relative to level tangents on rural two-lane highways were developed from the models for each of the five combinations of horizontal and vertical alignment to present the results in a form suitable for incorporation in the HSM. This report should be of interest to safety engineers and planners as well as other AASHTO HSM users.

Monique R. Evans  
Director, Office of Safety  
Research and Development

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16. Abstract This report summarizes the results of research undertaken to quantify the safety effects of five types of horizontal and vertical alignment combinations for rural two-lane highways. The research is based on Federal Highway Administration Highway Safety Information System data for Washington State, including crash records from 2003 to 2008. The outcome is a set of safety prediction models for fatal and injury and property damage only crashes.  To present the results in a form suitable for incorporation in the American Association of State Highway and Transportation Officials <i>Highway Safety Manual</i> , crash modification factors representing safety performance relative to level tangents were developed from these models for each of the five combinations of horizontal and vertical alignment—horizontal curves and tangents on straight grades, horizontal curves and tangents at type 1 crest vertical curves, horizontal curves and tangents at type 1 sag vertical curves, horizontal curves and tangents at type 2 crest vertical curves, and horizontal curves and tangents at type 2 sag vertical curves. <sup>(1)</sup>			
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# SI\* (MODERN METRIC) CONVERSION FACTORS

## APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
<b>LENGTH</b>				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
<b>AREA</b>				
in <sup>2</sup>	square inches	645.2	square millimeters	mm <sup>2</sup>
ft <sup>2</sup>	square feet	0.093	square meters	m <sup>2</sup>
yd <sup>2</sup>	square yard	0.836	square meters	m <sup>2</sup>
ac	acres	0.405	hectares	ha
mi <sup>2</sup>	square miles	2.59	square kilometers	km <sup>2</sup>
<b>VOLUME</b>				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft <sup>3</sup>	cubic feet	0.028	cubic meters	m <sup>3</sup>
yd <sup>3</sup>	cubic yards	0.765	cubic meters	m <sup>3</sup>
NOTE: volumes greater than 1000 L shall be shown in m <sup>3</sup>				
<b>MASS</b>				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
<b>TEMPERATURE (exact degrees)</b>				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
<b>ILLUMINATION</b>				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m <sup>2</sup>	cd/m <sup>2</sup>
<b>FORCE and PRESSURE or STRESS</b>				
lbf	poundforce	4.45	newtons	N
lbf/in <sup>2</sup>	poundforce per square inch	6.89	kilopascals	kPa

## APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
<b>LENGTH</b>				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
<b>AREA</b>				
mm <sup>2</sup>	square millimeters	0.0016	square inches	in <sup>2</sup>
m <sup>2</sup>	square meters	10.764	square feet	ft <sup>2</sup>
m <sup>2</sup>	square meters	1.195	square yards	yd <sup>2</sup>
ha	hectares	2.47	acres	ac
km <sup>2</sup>	square kilometers	0.386	square miles	mi <sup>2</sup>
<b>VOLUME</b>				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m <sup>3</sup>	cubic meters	35.314	cubic feet	ft <sup>3</sup>
m <sup>3</sup>	cubic meters	1.307	cubic yards	yd <sup>3</sup>
<b>MASS</b>				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
<b>TEMPERATURE (exact degrees)</b>				
°C	Celsius	1.8C+32	Fahrenheit	°F
<b>ILLUMINATION</b>				
lx	lux	0.0929	foot-candles	fc
cd/m <sup>2</sup>	candela/m <sup>2</sup>	0.2919	foot-Lamberts	fl
<b>FORCE and PRESSURE or STRESS</b>				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in <sup>2</sup>

\*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.  
(Revised March 2003)

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## CHAPTER 1—INTRODUCTION

This chapter presents an introduction to the report, including key background information for the research, research objectives and scope, and the organization of this report.

### BACKGROUND

The safety effects of horizontal curves and grades on highways have been quantified separately, but it is not currently known whether and how the safety performance of horizontal curves and grades interact. Furthermore, there are established safety effects for crest and sag vertical curves, and it is not known whether and how the safety performance of crest and sag vertical curves interacts with any horizontal curves that may be present. The research presented in this report was undertaken as part of the Federal Highway Administration (FHWA) Highway Safety Information System (HSIS) contract to investigate these issues and provide results in a form useful to highway designers and highway safety engineers.

### RESEARCH OBJECTIVES AND SCOPE

The objective of the research was to quantify the safety effects of horizontal and vertical alignment combinations and to present the results in a form suitable for incorporation in the American Association of State Highway and Transportation Officials (AASHTO) *Highway Safety Manual* (HSM).<sup>(1)</sup>

The scope of the work initially included horizontal and vertical alignment for the four facility types whose safety performance is addressed in the first edition of the HSM:<sup>(1)</sup>

- Rural two-lane highways.
- Rural multilane undivided highways.
- Rural multilane divided highways.
- Urban and suburban arterials.

The research found that only rural two-lane highways had sufficient data for which modeling efforts appeared promising.

### REPORT ORGANIZATION

The remainder of this report is organized as follows:

- Chapter 2 reviews the existing state of knowledge about the safety effects of horizontal curves and grades prior to the current research.
- Chapter 3 describes the HSIS database used for analysis.

- Chapter 4 discusses the statistical analysis approach and presents the results of the safety analysis.
- Chapter 5 details how crash modification factors (CMFs) are derived.
- Chapter 6 presents the conclusions and recommendations.

## CHAPTER 2—STATE OF KNOWLEDGE LITERATURE REVIEW

This chapter reviews the existing state of knowledge about the safety effects of horizontal curves and grades prior to the current research.

### GEOMETRIC DESIGN CRITERIA

Design criteria for horizontal and vertical alignment are presented in chapter 3 of the AASHTO *Policy on Geometric Design of Highways and Streets*, commonly known as the *Green Book*.<sup>(2)</sup>

Many State highway agencies have their own design manuals, but they tend to closely resemble the AASHTO *Green Book* in the areas of horizontal and vertical alignment.<sup>(2)</sup>

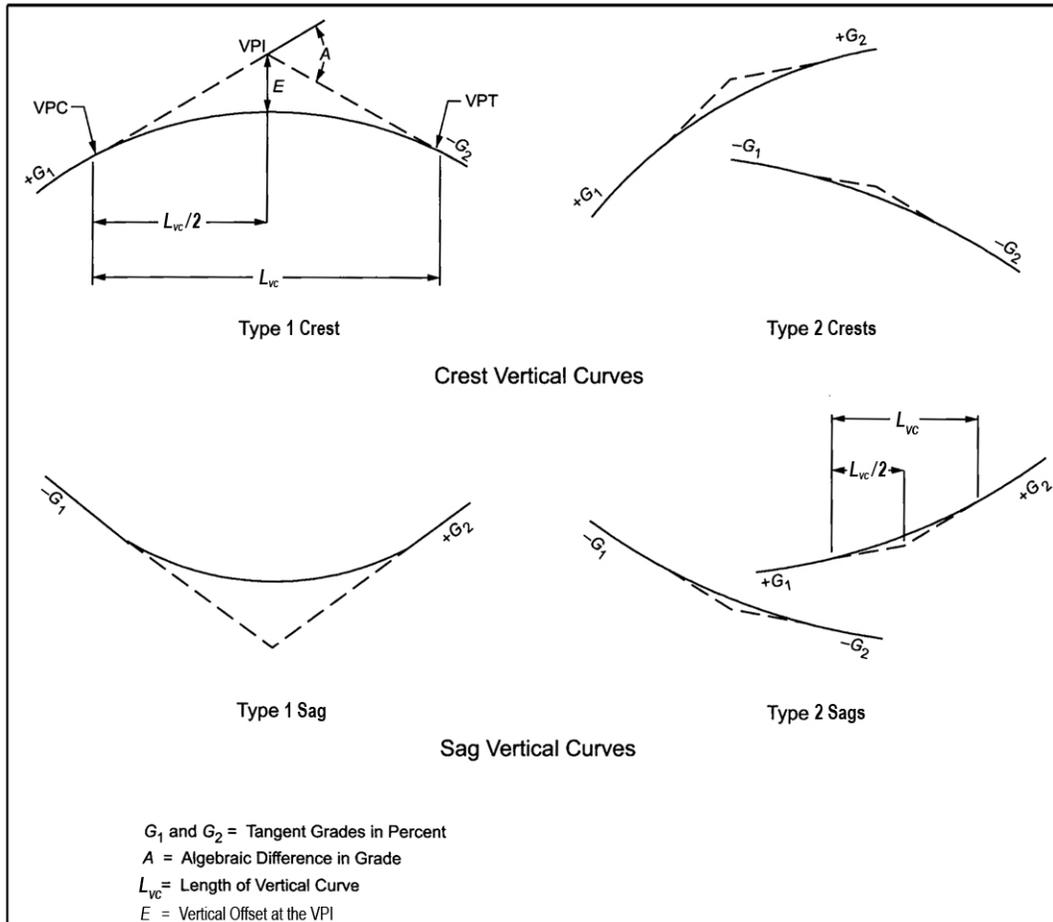
The key design parameters for horizontal curves include the following:

- Radius of curvature.
- Length of curve.
- Superelevation.
- Transition design.

Straight road sections with no horizontal curvature are generally referred to as “tangents” because they are usually tangent to any horizontal curves that they adjoin.

The fundamental design parameter for vertical alignment is the *percent grade*, which is defined as 100 times the change in roadway elevation per unit length of roadway measured in the horizontal plane. A road section with constant percent grade, regardless of its horizontal alignment, is generally referred to as a straight grade. Where the grade of the roadway changes, the straight grade sections are normally joined by a parabolic vertical curve. Figure 1 illustrates the four types of vertical curves (two types of crest vertical curves and two types of sag vertical curves) that are used in highway design. Key design parameters for vertical curves include the following:

- Algebraic difference in grade.
- Length of curve.
- Ratio of algebraic difference in grade and length of curve ( $K$ ), which represents the sharpness of the vertical curve.



Source: AASHTO. Used by permission.

**Figure 1. Illustration. Types of vertical curves.**<sup>(2)</sup>

### SAFETY EFFECTS OF HORIZONTAL ALIGNMENT

The most widely used safety relationship for horizontal curves on rural two-lane highways was developed by Zegeer et al. in the early 1990s as shown in figure 2\*:<sup>(3,4)</sup>

$$N = [1.55(L_c)(V) + 0.014(D)(V) - 0.012(S)(V)](0.978)^{W-30}$$

**Figure 2. Equation. Relationship between crash rates and curve characteristics.**

Where:

$N$  = Total number of crashes on curve per year.

$L_c$  = Length of curve (mi).

$V$  = Volume of vehicles (in millions) passing through the curve in a 1-year period.

$D$  = Degree of curvature ( $5,730/\text{radius of curvature (ft)}$ ).

$S$  = Presence of spiral transitions, where  $S = 0$  if no spirals exist and  $S = 1$  if spirals do exist.

$W$  = Width of roadway on curve (ft).

$D$ , which represents the deflection angle for the curve per 100 ft of curve length, is an obsolete measure that is no longer used in the AASHTO *Green Book*.<sup>(2)</sup> Therefore, the equivalent to degree of curvature expressed in terms of the radius of curvature is provided. The original Zegeer et al. model was expressed in terms of predicting crash frequency for a 5-year period; the model in figure 2 is expressed on a per-year basis for ease of comparison to other models.

No safety relationships for horizontal curvature have been developed for highway types other than rural two-lane highways, although such relationships for freeways have been developed in National Cooperative Highway Research Program Project 17-45.<sup>(5)</sup>

An assessment of literature conducted by Harwood, et al., as part of the development of the FHWA Interactive Highway Safety Design Model, concluded that the Zegeer, et al. model in figure 2 was the most useful and accurate model to account for the safety effect of horizontal curves on rural two-lane highways.<sup>(6,7)</sup> Harwood et al. expressed the Zegeer, et al. model as a CMF shown in figure 3\*:<sup>(6)</sup>

$$CMF_{HC} = \frac{1.55L_C + \frac{80.2}{R} - 0.012S}{1.55L_C}$$

**Figure 3. Equation. CMF for horizontal curves on rural two-lane highways.**

Where:

$CMF_{HC}$  = CMF for horizontal curvature on a rural two-lane highway.

$R$  = Radius of curvature (ft).

The base condition for this CMF is a tangent roadway. The AASHTO HSM adopted the CMF in figure 3 to represent the safety effects of horizontal curvature on rural two-lane highways.<sup>(1)</sup>

The AASHTO HSM also includes a CMF for the safety effect of superelevation for horizontal curves of rural two-lane highways as a function of superelevation variance as shown in figure 4\*:

$$\begin{aligned} CMF_{SV} &= 1.00 \text{ for } SV < 0.01 \\ CMF_{SV} &= 1.00 + 6(SV - 0.01) \text{ for } 0.01 \leq SV < 0.02 \\ CMF_{SV} &= 1.06 + 3(SV - 0.02) \text{ for } SV \geq 0.02 \end{aligned}$$

**Figure 4. Equation. CMF for superelevation as a function of superelevation variance.**

Where:

$CMF_{SV}$  = CMF for superelevation variance on a rural two-lane highway.

$SV$  = Superelevation variance (ft/ft), which represents the design superelevation rate presented in the AASHTO *Green Book* minus the actual superelevation of the curve.<sup>(2)</sup>

CMF was also adapted by Harwood et al. from the work of Zegeer et al.<sup>(3,4,6)</sup> The base condition for CMF is a horizontal curve with superelevation within 0.01 ft/ft of the applicable design superelevation presented in the AASHTO *Green Book*.<sup>(2)</sup>

No CMFs for horizontal curvature on rural multilane undivided highways, rural multilane divided highways, or urban and suburban arterials are included in the AASHTO HSM.<sup>(1)</sup>

## SAFETY EFFECTS OF VERTICAL ALIGNMENT

Table 1 presents the AASHTO HSM CMF, denoted as  $CMF_{5r}$ , which represents the safety effect of percent grade on rural two-lane highways.

**Table 1. CMF for grade of roadway segments.<sup>(1)</sup>**

Level Grade ( $\leq 3$ percent)	Moderate Terrain (3 percent < Grade $\leq 6$ percent)	Steep Terrain ( $> 6$ percent)
1.00	1.10	1.16

The base condition for the CMFs shown in table 1 is a level roadway. Figure 5 represents the underlying functional form for CMF shown in table 1.

$$CMF_G = 1.016^{abs(G)}$$

**Figure 5. Equation. CMF for percent grade on rural two-lane highways.**

Where:

$CMF_G$  = CMF for percent grade on a rural two-lane highway.

$abs(G)$  = Absolute value of percent grade.

The CMF in this form, as a continuous function, is a more useful representation of the safety effect of percent grade than the form shown in table 1, as the stepwise function in table 1 may be misleading.

The CMF for percent grade shown in table 1 and figure 5 applies only to straight grades. There are no CMFs in the AASHTO HSM for crest or sag vertical curves on rural two-lane highways. Also, there are no vertical alignment CMFs in the AASHTO HSM for facility types other than rural two-lane highways.

## INTERACTIONS BETWEEN SAFETY EFFECTS OF HORIZONTAL AND VERTICAL ALIGNMENT

While the AASHTO HSM provides CMFs for the safety effects of horizontal curvature and percent grade on rural two-lane highways, it does not have any method for accounting for the interactions between these effects. In other words, in the AASHTO HSM procedures for rural two-lane highways, the safety effect of a horizontal curve is the same whether it is located on a level roadway, a straight grade, or a vertical curve. Similarly, the safety effect of a straight grade is the same whether it is located on a tangent roadway or on a horizontal curve. Researchers have always assumed that there are interactions between the safety effects of horizontal and vertical alignment, but this has not been demonstrated in a form useful for safety prediction. Recent research by Easa and You have partially addressed this issue with separate models for horizontal curves and horizontal tangents, but they did not tie their models back to a common base condition (such as a level tangent roadway) or express the modeling results in a form that could be considered a CMF.<sup>(8,9)</sup>

## CHAPTER 3—DATABASE DESCRIPTION

This chapter describes the database used in the analysis. The research was performed with HSIS data for State highways in Washington. This is the only data source that includes system-wide data on curve and grade geometry that can be linked to system-wide roadway characteristics, traffic volume, and crash data.

### DATABASE DEVELOPMENT

Research began with a review of available databases that contain roadway data (including horizontal and vertical alignment), traffic volume data, and crash data in a format that could be linked by location, with a primary focus on available HSIS data. The only dataset found with sufficient detail concerning horizontal and vertical alignment was HSIS data for State highways in Washington.

Preliminary data processing was performed to identify the following types of roadways on the Washington State highway system (note that the abbreviations are used in table 2 through table 5):

- Rural two-lane highways (R2U).
- Rural four-lane undivided highways (R4U).
- Rural four-lane divided highways (nonfreeways) (R4D).
- Urban/suburban two-lane undivided arterials (U2U).
- Urban/suburban three-lane arterials with a center two-way left-turn lane (U3T).
- Urban/suburban four-lane undivided arterials (U4U).
- Urban/suburban four-lane divided arterials (nonfreeways) (U4D).
- Urban/suburban five-lane arterials with a center two-way left-turn lane (U5T).

Roadway segments with atypical features such as passing and climbing lanes as well as roadway segments with transitions between grades identified as angle points were eliminated from consideration. Angle points most likely represent crest or sag vertical curves that were too short or not well enough defined to be measured properly. Finally, a limited set of roadways with obvious data problems, such as successive vertical curves whose lengths appeared to overlap, were also eliminated from consideration. The descriptive statistics for the roadway data shown below are based on the roadway lengths that were retained for analysis after the initial screening.

Next, each roadway segment was classified into categories by its horizontal and vertical alignment. Horizontal alignment was classified as follows:

- Tangent roadways.

- Roadways on horizontal curves.

Vertical alignment was classified as follows:

- Level roadways.
- Straight grades (constant percent grade of 1 percent or more).
- Type 1 crest vertical curves.
- Type 2 crest vertical curves.
- Type 1 sag vertical curves.
- Type 2 sag vertical curves.

Figure 1 illustrates the distinction between types 1 and 2 crest and sag vertical curves. Every roadway segment was defined by its horizontal alignment, vertical alignment, and combination of horizontal and vertical alignments. Since there were two horizontal alignment categories and six vertical alignment categories, there were a total of 12 possible combinations of horizontal and vertical alignments. Where horizontal and vertical curves overlap, their beginnings and ends may not coincide; therefore, a new roadway segment began at any point where the horizontal or vertical alignment changed. Thus, some segments might include all of a horizontal or vertical curve, while others might include only part of a horizontal or vertical curve. The length of every roadway segment was determined for use in the analysis as well as the length of any horizontal curve that was wholly or partially within the segment and the length of any vertical curve ( $L_{VC}$ ) that was wholly or partially within the segment. Additionally, each horizontal curve was characterized by its radius. No data on the superelevation of horizontal curves were available for analysis. Each straight grade was characterized by its percent grade ( $G$ ). Each vertical curve was characterized by its approach grade ( $G_1$ ) and departure grade ( $G_2$ ), its algebraic difference in grade [ $A = \text{abs}(G_1 - G_2)$ ]\*, and the ratio of its length to its algebraic difference in grade ( $K = L_{VC}/A$ ).

Crash data for a 6-year period (2003 to 2008) were obtained and used in the analysis. Each crash was assigned to a particular roadway segment, with particular horizontal and vertical alignment based on its assigned milepost location. Since the results of this research are intended for use in the roadway segment procedures of the AASHTO HSM, only nonintersection crashes were considered. Nonintersection crashes are those that did not occur at an intersection and were not classified by the investigating officer or data coder as related to the operation of an intersection. The traffic volume for each roadway segment was determined from available traffic volume data.

Some of the roadway segments included in the analysis may have contained minor intersections, but crashes occurring at or related to those intersections have been excluded from the analysis. No major intersections were included within a roadway segment because a change in traffic volume along the roadway would have led to the creation of a roadway segment boundary.

Driveway crashes are considered roadway segment crashes. As a result, they were included in the analysis. The amount of driveway access within segments may have varied, but all of the segments were located in areas classified as rural.

## DESCRIPTIVE STATISTICS

The following discussion presents descriptive statistics for the roadway sections available for analysis including roadway length (miles), exposure [million vehicle miles traveled in the 6-year period (MVMT)]\*, crash frequencies, and crash rates per MVMT for specific combinations of horizontal and vertical alignment and roadway type. Roadway length, exposure, crash frequency, and crash rate are presented in table 2 through table 5, respectively.

Of the 6,944 mi of roadway in the entire Washington HSIS database, 4,785 mi (69 percent) are on rural two-lane highways. Of these, 3,457 mi were used for analysis. Rural two-lane highways with passing or climbing lanes and segments with missing or obviously incorrect alignment data (e.g., overlapping curves) were excluded from the study. Therefore, based on data availability, the analysis of the safety effects of horizontal curve and grade combinations in this report focused on rural two-lane highways only.

**Table 2. Roadway length (mi) in available data from Washington HSIS database.**

Alignment Type	Roadway Element	Rural Highways			Urban/Suburban Arterials				
		R2U	R4D	R4U	U2U	U3T	U4D	U4U	U5T
Horizontal	Tangent	2,472.1	122.4	4.9	122.1	15.9	24.2	46.3	48.0
	Curve	985.0	54.0	1.2	43.3	2.5	6.4	13.1	8.9
	<b>Total</b>	<b>3,457.1</b>	<b>176.4</b>	<b>6.1</b>	<b>165.4</b>	<b>18.4</b>	<b>30.7</b>	<b>59.4</b>	<b>56.9</b>
Vertical	Straight grade	2,260.7	107.6	4.0	112.4	12.2	17.6	37.5	36.2
	Type 1 crest	364.5	19.9	0.1	14.7	1.7	4.5	6.0	6.3
	Type 2 crest	300.8	20.1	0.5	13.3	2.0	2.7	6.1	5.5
	Type 1 sag	252.1	12.8	0.5	12.0	1.2	3.7	3.9	5.1
	Type 2 sag	279.1	16.0	0.9	13.0	1.3	2.2	5.9	3.8
	<b>Total</b>	<b>3,457.1</b>	<b>176.4</b>	<b>6.1</b>	<b>165.4</b>	<b>18.4</b>	<b>30.7</b>	<b>59.4</b>	<b>56.9</b>

**Table 3. Exposure (MVMT) in available data from Washington HSIS database (2003–2008).**

Alignment Type	Roadway Element	Rural Highways			Urban/Suburban Arterials				
		R2U	R4D	R4U	U2U	U3T	U4D	U4U	U5T
Horizontal	Tangent	16,675.2	3,648.2	113.5	3,228.2	483.5	1,408.2	2,613.9	2,826.3
	Curve	6,194.2	1,587.6	35.1	1,108.8	76.5	351.3	684.0	500.3
	<b>Total</b>	<b>22,869.5</b>	<b>5,235.8</b>	<b>148.6</b>	<b>4,336.9</b>	<b>560.0</b>	<b>1,759.5</b>	<b>3,297.9</b>	<b>3,326.5</b>
Vertical	Straight grade	14,847.0	3,216.5	91.6	2,894.0	366.8	991.4	2,072.1	2,091.4
	Type 1 crest	2,616.4	636.6	2.3	398.3	54.3	265.9	341.4	375.6
	Type 2 crest	1,870.5	521.2	11.9	364.1	62.4	162.4	341.5	322.7
	Type 1 sag	1,772.6	392.6	23.4	344.5	38.0	209.3	227.4	309.4
	Type 2 sag	1,762.9	469.0	19.4	336.0	38.5	130.5	315.5	227.4
	<b>Total</b>	<b>22,869.5</b>	<b>5,235.8</b>	<b>148.6</b>	<b>4,336.9</b>	<b>560.0</b>	<b>1,759.5</b>	<b>3,297.9</b>	<b>3,326.5</b>

**Table 4. Fatal and injury (FI), property damage only (PDO), and total 6-year crash frequencies in available data from Washington HSIS database.**

Alignment Type	Roadway Element	Rural Highways			Urban/Suburban Arterials				
		R2U	R4D	R4U	U2U	U3T	U4D	U4U	U5T
<b>FI Crash Frequencies in 6 Years (2003–2008)</b>									
Horizontal	Tangent	7,360	865	77	2,564	335	1,557	5,100	2,867
	Curve	3,659	353	11	772	55	406	1,017	245
	<b>Total</b>	<b>11,019</b>	<b>1,218</b>	<b>88</b>	<b>3,336</b>	<b>390</b>	<b>1,963</b>	<b>6,117</b>	<b>3,112</b>
Vertical	Straight grade	7,347	740	63	2,313	260	1,291	3,570	2,117
	Type 1 crest	1,168	125	1	260	25	262	634	323
	Type 2 crest	826	97	5	292	39	156	780	222
	Type 1 sag	896	108	4	235	29	101	577	304
	Type 2 sag	782	148	15	236	37	153	556	146
	<b>Total</b>	<b>11,019</b>	<b>1,218</b>	<b>88</b>	<b>3,336</b>	<b>390</b>	<b>1,963</b>	<b>6,117</b>	<b>3,112</b>
<b>PDO Crash Frequencies in 6 Years (2003–2008)</b>									
Horizontal	Tangent	10,519	1,403	213	3,753	519	2,841	9,012	4,323
	Curve	4,758	621	14	1,264	77	741	1,800	426
	<b>Total</b>	<b>15,277</b>	<b>2,024</b>	<b>227</b>	<b>5,017</b>	<b>596</b>	<b>3,582</b>	<b>10,812</b>	<b>4,749</b>
Vertical	Straight grade	10,222	1,273	185	3,451	401	2,259	6,250	3,315
	Type 1 crest	1,498	229	10	397	55	527	1,208	461
	Type 2 crest	1,264	162	4	437	64	310	1,390	333
	Type 1 sag	1,154	177	13	383	37	227	1,037	402
	Type 2 sag	1,139	183	15	349	39	259	927	238
	<b>Total</b>	<b>15,277</b>	<b>2,024</b>	<b>227</b>	<b>5,017</b>	<b>596</b>	<b>3,582</b>	<b>10,812</b>	<b>4,749</b>
<b>Total Crash Frequencies in 6 Years (2003–2008)</b>									
Horizontal	Tangent	17,879	2,268	290	6,317	854	4,398	14,112	7,190
	Curve	8,417	974	25	2,036	132	1,147	2,817	671
	<b>Total</b>	<b>26,296</b>	<b>3,242</b>	<b>315</b>	<b>8,353</b>	<b>986</b>	<b>5,545</b>	<b>16,929</b>	<b>7,861</b>
Vertical	Straight grade	17,569	2,013	248	5,764	661	3,550	9,820	5,432
	Type 1 crest	2,666	354	11	657	80	789	1,842	784
	Type 2 crest	2,090	259	9	729	103	466	2,170	555
	Type 1 sag	2,050	285	17	618	66	328	1,614	706
	Type 2 sag	1,921	331	30	585	76	412	1,483	384
	<b>Total</b>	<b>26,296</b>	<b>3,242</b>	<b>315</b>	<b>8,353</b>	<b>986</b>	<b>5,545</b>	<b>16,929</b>	<b>7,861</b>

**Table 5. FI, PDO, and total crash rates per MVMT in available data from Washington HSIS database.**

Alignment Type	Roadway Element	Rural Highways			Urban/Suburban Arterials				
		R2U	R4D	R4U	U2U	U3T	U4D	U4U	U5T
<b>FI Crash Rate per MVMT</b>									
Horizontal	Tangent	0.441	0.237	0.679	0.794	0.693	1.106	1.951	1.014
	Curve	0.591	0.222	0.313	0.696	0.719	1.156	1.487	0.490
Vertical	Straight grade	0.495	0.230	0.688	0.799	0.709	1.302	1.723	1.012
	Type 1 crest	0.446	0.196	0.432	0.653	0.460	0.985	1.857	0.860
	Type 2 crest	0.442	0.186	0.421	0.802	0.625	0.961	2.284	0.688
	Type 1 sag	0.505	0.275	0.171	0.682	0.763	0.483	2.537	0.982
	Type 2 sag	0.444	0.316	0.775	0.702	0.961	1.173	1.762	0.642
<b>PDO Crash Rate per MVMT</b>									
Horizontal	Tangent	0.631	0.385	1.877	1.163	1.073	2.017	3.448	1.530
	Curve	0.768	0.391	0.399	1.140	1.007	2.110	2.632	0.852
Vertical	Straight grade	0.688	0.396	2.020	1.192	1.093	2.279	3.016	1.585
	Type 1 crest	0.573	0.360	4.323	0.997	1.012	1.982	3.539	1.227
	Type 2 crest	0.676	0.311	0.337	1.200	1.025	1.909	4.070	1.032
	Type 1 sag	0.651	0.451	0.555	1.112	0.974	1.085	4.560	1.299
	Type 2 sag	0.646	0.390	0.775	1.039	1.013	1.985	2.938	1.047
<b>Total Crash Rate per MVMT</b>									
Horizontal	Tangent	1.072	0.622	2.556	1.957	1.766	3.123	5.399	2.544
	Curve	1.359	0.614	0.712	1.836	1.726	3.265	4.118	1.341
Vertical	Straight grade	1.183	0.626	2.707	1.992	1.802	3.581	4.739	2.597
	Type 1 crest	1.019	0.556	4.755	1.649	1.472	2.967	5.396	2.087
	Type 2 crest	1.117	0.497	0.758	2.002	1.650	2.870	6.354	1.720
	Type 1 sag	1.156	0.726	0.726	1.794	1.737	1.567	7.097	2.282
	Type 2 sag	1.090	0.706	1.550	1.741	1.975	3.158	4.701	1.689

Note: Crash rates cannot be added; therefore, no totals are shown.

The majority of rural two-lane roadway segments included in the analysis experienced no crashes in the 6-year period. Table 6 shows the percentage of roadway segments with no crashes and with one or more crashes for each roadway and severity type. The data clearly show the highly skewed crash distribution for all severity types and all roadway types. The high percentages of roadway segments without crashes (78 to 89 percent for FI crashes and 72 to 84 percent for PDO crashes) provided a challenge in modeling crash frequencies.

**Table 6. Percent of sections with and without crashes on rural two-lane highways in available data from Washington HSIS database (2003–2008).**

Vertical and Horizontal Alignment Combination		FI Crashes		PDO Crashes		Total Crashes	
		Percent of Sections With:					
		Zero Crashes	1+ Crashes	Zero Crashes	1+ Crashes	Zero Crashes	1+ Crashes
Straight grade	Horizontal curve	84	16	81	19	72	28
	Tangent on nonlevel grade	84	16	79	21	71	29
Type 1 crest	Horizontal curve	84	16	82	18	73	27
	Tangent on nonlevel grade	84	16	80	20	72	28
Type 2 crest	Horizontal curve	88	12	84	16	78	22
	Tangent on nonlevel grade	89	11	84	16	78	22
Type 1 sag	Horizontal curve	86	14	84	16	76	24
	Tangent on nonlevel grade	86	14	83	17	75	25
Type 2 sag	Horizontal curve	86	14	84	16	76	24
	Tangent on nonlevel grade	86	14	83	17	75	25
Level tangent	Level tangent	78	22	72	28	63	37

### VALUE RANGE OF ROADWAY CHARACTERISTICS

Prior to statistical modeling, the parameters of interest were assessed for extreme values (both high and low). This was done using a combination of plots of crash rates per MVMT versus selected parameters and distributions of the individual parameters. The following rules were implemented:

- Roadway segments less than 0.01 mi long were excluded from analysis (such short segments likely represent small overlaps in horizontal and vertical features and are unlikely to be useful analysis sections).
- For type 1 crest and type 1 sag vertical curves, segments where both initial ( $G_1$ ) and final ( $G_2$ ) grades were less than 1 percent in absolute value were excluded (such minor vertical curves are very close to being level).
- For type 2 crest and type 2 sag vertical curves, segments where  $A = abs(G_1 - G_2)$  was less than 1 percent were excluded (such minor vertical curves are very close to being straight grades).
- All records with  $K$  exceeding 1,000 were excluded (these were typically long, vertical curves with small grade changes and could be classified as straight grades, including level grades).
- All records with a curve radius exceeding 11,460 ft were excluded (these could be classified as tangents for all practical purposes).
- Horizontal curves with a radius less than 100 ft were included in the analysis, but the radius was set at 100 ft based on guidance in the AASHTO HSM.<sup>(1)</sup>

## CHAPTER 4—STATISTICAL ANALYSIS

The overall statistical approach to estimating the safety effects of horizontal curve and grade combinations on rural two-lane highways is presented in this chapter along with the results for each type of combination for FI and PDO crashes.

### ANALYSIS APPROACH

The safety effects of horizontal curve and grade combinations are estimated based on a cross-sectional analysis using a generalized linear model approach assuming a negative binomial (NB) distribution of crash counts and an exponential model using the combined crash data from all 6 years and selected roadway geometrics. FI and PDO crashes were modeled separately for each of the five types of horizontal curve and grade combinations.

### Selection of Independent Variables Considered in Models

The parameters considered in each model may include the following:

- Average annual daily traffic (AADT) (averaged across all 6 years).
- Segment length.
- Horizontal curve radius.
- Absolute value of percent grade.
- Horizontal curve length.
- Vertical curve length.
- Algebraic difference between the initial and final grades ( $A$ ).
- Measure of the sharpness of vertical curvature ( $K$ ).
- Relevant interactions of selected parameters.

For each type of horizontal curve and grade combination, the dataset used for modeling included the roadway segments for the relevant curve and grade combination but also all level tangents (i.e., no horizontal curvature and grade < 1 percent) to serve as the base condition.

Modeling was performed by encoding each parameter, or a transformation of it, so as to treat roadway segments on horizontal curve and grade combinations separately from tangent roadways on nonlevel grades and from level tangents (base condition). Segments for all of these types were used to develop a single model. This is equivalent to using indicator variables for each type of roadway segment. For example, since the natural log of the inverse radius was used in some of the models, the radius was recoded as  $\ln(2 \times \frac{5,730}{R})$  based on the following reasoning:

the smallest value of  $\frac{5,730}{R}$  for horizontal curves in the database is 0.5. Therefore, the ratio,  $\frac{5,730}{R}$ , was multiplied by 2 such that the smallest value of  $\ln(2 \times \frac{5,730}{R})$  becomes zero. This term was then set equal to zero for all tangents. This approach ultimately allows for the computation of CMFs using the developed models (discussed in further detail in chapter 5 of this report).

Within each vertical alignment type (i.e., straight grades, type 1 crest vertical curves, type 1 sag vertical curves, type 2 crest vertical curves, and type 2 sag vertical curves), the functional form of the relationship between crash frequency (FI or PDO) and the parameters listed was assessed separately. The following approach was used to explore the appropriate functional form for these relationships:

- Each parameter, other than AADT, was categorized into three groups, which were typically of equal size (tertiles).
- A crash prediction model was developed including AADT and only the interaction of all categorized parameters [this is a standard analysis of variance (ANOVA) using an NB distribution and a log link]\*.
- The safety effect of one parameter was plotted against the cell means of another parameter, encoding the data by the levels of the third parameter. If a four-way interaction was included, then multiple sets of plots were generated.
- The shape of the relationship between safety effects and a given parameter across the levels of another parameter was assessed.
- These trends were assessed for each model to determine whether they were consistent. If not, an assessment was made to determine whether interactions exist.

Based on the visual assessment of these relationships, a final model form was selected using all parameters and relevant interactions. The parameters in these final model forms were continuous variables. In other words, the categorization into three groups was used only for exploring the potential functional forms for the data and was not used in the final models.

Before analyzing crashes on horizontal curves, tangents on nonlevel grades, and level tangents using a single model, the effect of vertical curve characteristics was assessed using tangents alone. This was done by comparing the effect of vertical curve characteristics on crashes/mi/year between level tangents and tangents on nonlevel grades for each type of vertical curve. This subset of data represents the largest proportion of the database. If a vertical curve parameter showed a statistically significant effect for tangents on nonlevel grades, then that parameter was included as such in the model using all three types of roadway (horizontal curves, level tangents, and tangents on nonlevel grades).

An attempt was also made to model crashes on horizontal curves and tangents by allowing separate intercepts and AADT slopes (i.e., an analysis of covariance) for each segment type to assess whether the relationship between crash frequency and AADT differs among horizontal

curves, tangents on nonlevel grades, and base condition tangents. This modeling effort was inconclusive (e.g., slopes were inconsistent and counterintuitive) and therefore abandoned.

Final crash prediction models were derived for horizontal curves and tangents using the same group of level tangent sections as the base condition for all five horizontal curve and grade combinations. A stepwise approach was used where all parameters and interactions were included. The least significant interaction(s) and then the least significant parameter(s) were eliminated one at a time until all remaining interactions and parameters were significant. This is known as backwards stepwise selection. At each step, extreme data points were excluded from the data using leverage estimates, residuals, or Cook's D criterion, all statistical criteria to evaluate the goodness-of-fit of the model to the data. In general, a 5 percent significance level associated with the type 3 chi-squared statistic was selected. All analyses were performed using a procedure for fitting generalized linear models of SAS<sup>®</sup> Version 9.3.<sup>(10)</sup>

It should be noted that additional geometric features for roadway segments, such as lane and shoulder widths, were not included in the analysis. The decision to exclude other geometric features was made because they were outside the scope of the current research. Experience with the Zegeer et al. results found that the roadway width term was dropped out of the final CMF.<sup>(3,4,6)</sup> Also, it was unlikely that the available data would support inclusion of additional terms.

### **Assessment of Goodness-of-Fit of Final Models**

Once a model for a specific horizontal and vertical alignment combination was finalized, its goodness-of-fit to the observed crash data was evaluated. For each alignment combination and severity level, predicted crash frequencies versus observed frequencies were plotted and assessed to determine how well the data followed the line of equality. However, it should be recognized that perfect or near perfect agreement between predicted and observed crash frequencies should not be expected no matter how good the methodology is. The predicted crash frequencies, at their best, represent an estimate of the long-term average crash frequency for similar roadway segments. The observed crash frequency is simply one observation from a random process whose mean is estimated by the predicted crash frequency. There is no reason to expect that one observation from a random process should exactly equal the long-term mean. In addition, the methodology cannot predict a crash frequency of zero because each model has a positive intercept. This is reasonable because no roadway segment can ever be expected to be crash-free in the long term. However, in any given time period, it is reasonable to expect that many roadway segments, particularly lower volume segments, will experience zero observed crashes.

To assess the goodness-of-fit of each model, the level of agreement between predicted and observed crash frequencies was estimated based on the percentage of extreme observations in the 6-year period at the 5 percent significance level. Consideration of the likelihood of extreme observed crash frequencies, either high or low, is a method that directly takes into account the parameters of the NB distribution (i.e., mean and dispersion) of crash frequencies at a particular type of roadway segment.

Let  $O_i$  denote the number of observed crashes of a given type (FI or PDO) on a roadway segment,  $i$ , during the 6-year period. The likelihood of observing  $O_i$  crashes in 6 years is then

computed under the assumption that  $O_i$  is an observation from a NB distribution with mean  $\mu_i$  and dispersion parameter  $k$ . The mean is the predicted number of crashes in the 6-year period calculated using the prediction model applicable for roadway segment  $i$ . The dispersion parameter is obtained when developing the final model.

The likelihood,  $p_i$ , of observing  $O_i$  or fewer crashes at roadway segment  $i$  can then be written as **shown in figure 6 and figure 7\***:

$$p_i = \text{prob}(\text{Number of crashes} \leq O_i)$$

**Figure 6. Equation. Probability of observing a given number of crashes.**

Or:

$$p_i = \sum_{x=0}^{O_i} f_i(x)$$

**Figure 7. Equation. Probability of observing a given number of crashes expressed as a cumulative distribution.**

Where  $f_i(x)$  is the probability distribution function for an NB with  $\mu_i$  and  $k$  and values of  $x = 0, 1, 2, \dots, O_i$ . If  $p_i \geq 0.5$ , then  $p_i$  was calculated as  $1 - p_i$ . This approach is equivalent to calculating the area under the cumulative distribution curve at either low or high tail of the distribution.

The following two final single-value criteria used to assess how well the observed crash frequencies can be estimated by the proposed methodology are proposed:

- The percent of roadway segments of a given alignment combination with observed crash frequencies outside the upper and lower 2.5 percentile of the theoretical distribution. This value,  $Pct_{unlikely}$ , is calculated as **shown in figure 8\***:

$$Pct_{unlikely} = 100 (\text{Number of segments where } p_i \leq 0.025 / \text{Total number of segments})$$

**Figure 8. Equation. Percent of roadway segments with unlikely low or high crash numbers.**

On average, the value of  $Pct_{unlikely}$  would be close to 5 percent if the methodology is to fit the observed data reasonably well because one would expect that on average, 5 percent of the observed values be outside the middle 95 percent of the distribution. Such observed crash frequencies would be considered extreme (low or high) crash frequencies under the estimated distribution.

- The average probability of more extreme than observed crash frequency. This is the average  $p_i$  across all segments  $i$  of a given roadway type for a particular crash type (FI or PDO). On average, the mean would be close to 0.25 if the methodology is to fit the observed data reasonably well.

## ANALYSIS RESULTS

This section presents the final modeling results for the five alignment categories for rural two-way highways. Each subsection is organized as follows:

- Basic description of database used.
- Final predictive regression equations.
- ANOVA tables.
- Goodness-of-fit results.

For each alignment category considered, a level tangent roadway serves as the base condition.

### Horizontal Curves and Tangents on Straight Grades

The following three alignment combinations shaded in figure 9 were included in the analysis:

- Horizontal curves on straight grades (including both level and nonlevel alignments) [number of roadway segments ( $N$ ) = 8,095 and total roadway length = 595 mi]\*.
- Tangents on nonlevel grades (grade  $\geq 1$  percent) ( $N$  = 7,569 and total roadway length = 727 mi).
- Level tangents—base condition (grade  $< 1$  percent) ( $N$  = 5,701 and total roadway length = 779 mi).

	Straight Grades		Vertical Curves			
	Level Abs(Grade) < 1%	Non level Abs(Grade) $\geq 1\%$	Type 1 Crest	Type 1 Sag	Type 2 Crest	Type 2 Sag
Tangents	<b>Base condition</b>					
Horizontal Curves						

Data in the three shaded cells were used in this analysis, while data in the blank cells were not used in this analysis.

**Figure 9. Illustration. Alignment combinations used in the analysis of horizontal curves and tangents on straight grades.**

Basic descriptive statistics such as sample size (i.e., number of roadway sections; total roadway length; and minimum, maximum, mean, and median values for specific parameters)\* are shown in table 7 for each of the three alignment types included in the analysis.

**Table 7. Descriptive statistics for horizontal curves and tangents on straight grades.**

Parameter	Minimum	Maximum	Mean	Median
<b>Horizontal Curves on Straight Grades</b> (N = 8,095; total roadway length = 595 mi)				
AADT (vehicles/day)	169	26,088	2,695	1,664
Section length (mi)	0.01	0.75	0.07	0.05
Horizontal curve length (mi)	0.01	1.19	0.15	0.11
Curve radius (ft)	100	11,459	2,067	1,433
Grade (percent)	0	9.67	2.11	1.53
FI crashes per MVMT	0	39.50	0.75	0
PDO crashes per MVMT	0	46.26	0.91	0
Total crashes per MVMT	0	54.62	1.66	0
<b>Tangents on Nonlevel Grades</b> (N = 7,569; total roadway length = 727 mi)				
AADT (vehicles/day)	169	26,088	2,700	1,644
Section length (mi)	0.01	0.99	0.10	0.06
Horizontal curve length (mi)				
Curve radius (ft)				
Grade (percent)	1.00	10.85	3.10	2.64
FI crashes per MVMT	0	39.33	0.61	0
PDO crashes per MVMT	0	44.14	0.80	0
Total crashes per MVMT	0	53.48	1.42	0
<b>Level Tangents—Base Condition</b> (N = 5,701; total roadway length = 779 mi)				
AADT (vehicles/day)	169	26,088	3,285	2,153
Section length (mi)	0.01	0.98	0.14	0.09
Horizontal curve length (mi)				
Curve radius (ft)				
Grade (percent)				
FI crashes per MVMT	0	34.21	0.46	0
PDO crashes per MVMT	0	39.50	0.67	0
Total crashes per MVMT	0	55.38	1.13	0

Note: No roadway segments exist in the shaded cells.

The final crash prediction models for FI and PDO crashes are as shown in figure 10 and figure 11\*:

$$N_{FI} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 G + b_3 \ln \left( 2 \times \frac{5,730}{R} \right) \times I_{HC} + b_4 \left( \frac{1}{R} \right) \left( \frac{1}{L_C} \right) \times I_{HC} \right]$$

**Figure 10. Equation. Predicted FI crashes on horizontal curves and tangents on straight grades (general form).**

$$N_{PDO} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 G + b_3 \ln \left( 2 \times \frac{5,730}{R} \right) \times I_{HC} + b_4 \left( \frac{1}{R} \right) \left( \frac{1}{L_C} \right) \times I_{HC} \right]$$

**Figure 11. Equation. Predicted PDO crashes on horizontal curves and tangents on straight grades (general form).**

Where:

$N_{FI}$  = FI crashes per mile per year.

$N_{PDO}$  = PDO crashes per mile per year.

AADT = Vehicles per day.

$G$  = Absolute value of percent grade (0 percent for level tangents;  $\geq 1$  percent otherwise).

$R$  = Curve radius (ft) (missing for tangents).

$I_{HC}$  = Horizontal curve indicator (1 for horizontal curves; 0 otherwise).

$L_C$  = Horizontal curve length (mi) (not applicable for tangents).

ln = Natural logarithm function.

$b_0, \dots, b_4$  = Regression coefficients.

The regression results, including the coefficient estimate, dispersion parameter, standard error, confidence limit, chi-squared statistic, and significance level for all statistically significant parameters and interaction are shown in table 8.

**Table 8. FI and PDO crash modeling results for horizontal curves and tangents on straight grades.**

Parameter Description	Regression Coefficient	Coefficient Estimate	Standard Error	Lower 95 Percent Confidence Limit	Upper 95 Percent Confidence Limit	Chi-Squared Statistic	Significance Level
<b>FI Crashes/Mi/Year</b>							
Intercept	$b_0$	-8.76	0.15	-9.05	-8.46	N/A	N/A
ln(AADT)	$b_1$	1.00	0.02	0.96	1.03	3,052.7	< 0.0001
Grade	$b_2$	0.044	0.01	0.03	0.06	27.5	< 0.0001
1/radius term <sup>a</sup>	$b_3$	0.19	0.02	0.16	0.22	116.3	< 0.0001
1/R $\times$ 1/ $L_C$ interaction	$b_4$	4.52	0.79	2.97	6.07	26.8	< 0.0001
Dispersion	N/A	0.85	0.04	0.77	0.94	N/A	N/A
<b>PDO Crashes/Mi/Year</b>							
Intercept	$b_0$	-8.63	0.14	-8.89	-8.36	N/A	N/A
ln(AADT)	$b_1$	1.03	0.02	1.00	1.06	4,003.5	< 0.0001
Grade	$b_2$	0.040	0.01	0.03	0.05	29.1	< 0.0001
1/radius term <sup>a</sup>	$b_3$	0.13	0.02	0.10	0.16	67.4	< 0.0001
1/R $\times$ 1/ $L_C$ interaction	$b_4$	3.80	0.84	2.15	5.45	17.3	< 0.0001
Dispersion	N/A	0.80	0.03	0.73	0.87	N/A	N/A

<sup>a</sup>1/radius term =  $\ln(2 \times 5,730/R)$ .

N/A = Not applicable.

Applying figure 8, the percentage of roadway segments with extremely high observed FI crash frequencies was 6.09 percent across all roadway segments, which was only slightly above the expected 5 percent. The percentages were 6.12 percent for segments on level tangents, 6.25 percent for segments on tangents on nonlevel grades, and 5.91 percent for segments on

horizontal curves on straight grades, indicating that a few roadway segments in these roadway categories experienced unusually high FI crash frequencies given the prediction model used. None of the segments experienced extremely low FI crash frequencies under the assumed model at the 5 percent significance level.

Similarly, the percentage of roadway segments with extremely high observed PDO crash frequencies was 6.56 percent across all roadway segments, which was slightly higher than that for FI crashes. The percentages were 6.71 percent for segments on level tangents, 6.90 percent for segments on tangents on nonlevel grades, and 6.14 percent for segments on horizontal curves on straight grades, indicating that a few roadway segments in these roadway categories experienced unusually high PDO crash frequencies given the prediction model used. None of the segments experienced extremely low PDO crash frequencies under the assumed model at the 5 percent significance level.

The average probability of predicting a more extreme than observed FI crash frequency was calculated using figure 7. Across all roadway segments, the average probability was 0.13. The average probability was 0.14 for segments on level tangents, 0.11 for segments on tangents on nonlevel grades, and 0.13 for segments on horizontal curves on straight grades, all considerably lower than the theoretically expected value of 0.25.

Similarly, the average probability of predicting a more extreme than observed PDO crash frequency was 0.14 across all roadway segments. The average probability was 0.16 for segments on level tangents, 0.13 for segments on tangents on nonlevel grades, and 0.15 for segments on horizontal curves on straight grades, all considerably lower than the theoretically expected value of 0.25. For both FI and PDO crashes, these low probabilities are an indication that the model might not provide an adequate fit to the data. This is not too surprising given the large number of roadway segments with a wide range of AADTs and geometrics and a high percentage of segments with zero crashes.

Substituting the regression coefficients in figure 10 and figure 11 with their corresponding estimates in table 8, the prediction models for FI and PDO crashes/mi/year are as shown in figure 12, figure 13, and figure 14\*:

$$N_{FI} = \exp \left[ -8.76 + 1.00 \ln(\text{AADT}) + 0.044G + 0.19 \ln \left( 2 \times \frac{5,730}{R} \right) \times I_{HC} + 4.52 \left( \frac{1}{R} \right) \left( \frac{1}{L_C} \right) \times I_{HC} \right]$$

**Figure 12. Equation. Predicted FI crashes on horizontal curves and tangents on straight grades (explicit form).**

$$N_{PDO} = \exp \left[ -8.63 + 1.03 \ln(\text{AADT}) + 0.040G + 0.13 \ln \left( 2 \times \frac{5,730}{R} \right) \times I_{HC} + 3.80 \left( \frac{1}{R} \right) \left( \frac{1}{L_C} \right) \times I_{HC} \right]$$

**Figure 13. Equation. Predicted PDO crashes on horizontal curves and tangents on straight grades (explicit form).**

$$N_{Total} = N_{FI} + N_{PDO}$$

**Figure 14. Equation. Predicted total crashes on horizontal curves and tangents on straight grades (general form).**

Table 8 clearly shows that AADT is the most predominant predictor of crashes as indicated by its high chi-squared statistic. Other parameters and interaction are one or two orders of magnitude smaller and indicate that once the variability due to traffic volume is accounted for, the remaining parameters explain only a small portion of the remaining variability in the data. This is a consistent trait across all models developed in this project.

These final models include only two significant parameters and an interaction from the list of parameters and interactions originally considered. This is in large part because all safety effects are estimated relative to level tangents as the base condition. Crash frequencies in this group of roadway segments alone (5,701 segments for a total of 779 mi) exhibit considerable variability (e.g., FI crash rates range from zero to 34.21 with a mean of 0.46 and a median of zero crashes per MVMT as shown in table 7). To detect a significant effect of any horizontal curve characteristic on straight grades, the effect of such a characteristic would need to be large relative to the variability in the base condition set. This, in effect, is the challenge in finding statistically significant safety effects of practical engineering relevance.

Figure 12 and figure 13 show that crash frequency increases with increasing percent grade and decreases with increasing curve radius, as expected. The interaction term between radius and curve length represents an additional effect on safety for short and sharp horizontal curves—as the radius decreases and the curve shortens, the last term in figure 12 and figure 13 increases, adding to the crash frequency. For long horizontal curves and curves with larger radii, this term approaches zero and thus will have little impact on the predicted crash frequency.

Of interest is the fact that the effects of percent grade, curve radius, and the interaction between radius and curve length is more pronounced for FI crashes than for PDO crashes.

Initial modeling effects indicated that there might be an interaction between horizontal curve radius and percent grade on straight grade segments, suggesting that the effect of curve radius on crash frequency might change with increasing percent grade. However, this effect was found to be an artifact of a few data points that were clearly outliers. These outliers were eliminated from the dataset used to produce the final models presented in figure 12 and figure 13. Thus, the analysis did not find a statistically significant interaction between horizontal curve radius and percent grade. The results of the study presented in this report are as follows:

- The results show an interaction between horizontal curve radius and length, such that short and sharp horizontal curves have particularly high crash frequencies.
- The effects of horizontal curve radius, horizontal curve length, and percent grade were fit in the same model rather than in separate research studies, as is the case with the curve and grade effects currently in the HSM.

### **Horizontal Curves and Tangents at Type 1 Crest Vertical Curves**

The following three alignment combinations shaded in figure 15 were included in the analysis:

- Horizontal curves at type 1 crest vertical curves ( $N = 1,219$  and total roadway length = 87 mi).

- Tangents at type 1 crest vertical curves ( $N = 2,089$  and total roadway length = 200 mi).
- Level tangents—base condition (grade < 1 percent) ( $N = 5,743$  and total roadway length = 833 mi).

	Straight Grades		Vertical Curves			
	Level Abs(Grade) <1 percent	Nonlevel Abs(Grade) ≥ 1 percent	Type 1 Crest	Type 1 Sag	Type 2 Crest	Type 2 Sag
Tangents	<b>Base Condition</b>					
Horizontal Curves						

Note: Data in the three shaded cells were used in this analysis, while data in the blank cells were not used in this analysis.

**Figure 15. Illustration. Alignment combinations used in the analysis of horizontal curves and tangents at type 1 crest vertical curves.**

Basic descriptive statistics such as sample size (i.e., number of roadway sections); total roadway length; and minimum, maximum, mean, and median values for specific parameters are shown in table 9 for each of the three alignment types included in the analysis.

**Table 9. Descriptive statistics for horizontal curves and tangents at type 1 crest vertical curves.**

Parameter	Minimum	Maximum	Mean	Median
<b>Horizontal Curves at Type 1 Crest Vertical Curves (N = 1,219; total roadway length = 87 mi)</b>				
AADT (vehicles/day)	175	26,088	3,059	1,877
Section length (mi)	0.01	0.72	0.07	0.06
Horizontal curve length (mi)	0.02	1.00	0.16	0.12
Curve radius (ft)	100	11,459	2,102	1,433
Vertical curve length (ft)	100	4,000	824	600
A	1.0	14.7	5.2	4.9
K	11.1	985.2	186.2	147.9
FI crashes per MVMT	0	23.10	0.55	0
PDO crashes per MVMT	0	28.12	0.66	0
Total crashes per MVMT	0	28.12	1.21	0
<b>Tangents at Type 1 Crest Vertical Curves (N = 2,089; nonlevel total roadway length = 200 mi)</b>				
AADT (vehicles/day)	169	26,088	3,105	1,858
Section length (mi)	0.01	0.59	0.10	0.08
Horizontal curve length (mi)				
Curve radius (ft)				
Vertical curve length (ft)	60	4,000	776	600

Parameter	Minimum	Maximum	Mean	Median
A	1.0	14.7	4.7	4.3
K	5.4	985.2	192.4	151.5
FI crashes per MVMT	0	20.85	0.40	0
PDO crashes per MVMT	0	25.43	0.57	0
Total crashes per MVMT	0	33.85	0.98	0
<b>Level Tangents—Base Condition</b>				
<b>(N = 5,743; total roadway length = 833 mi)</b>				
AADT (vehicles/day)	169	26,088	3,287	2,160
Section length (mi)	0.01	2.10	0.15	0.09
Horizontal curve length (mi)				
Curve radius (ft)				
Vertical curve length (ft)				
A				
K				
FI crashes per MVMT	0	34.21	0.46	0
PDO crashes per MVMT	0	39.50	0.67	0
Total crashes per MVMT	0	39.50	1.13	0

Note: No roadway segments exist in the shaded cells.

The final crash prediction models for FI and PDO crashes are shown in figure 16 and figure 17\*:

$$N_{FI} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 \left( \frac{5,730}{R} \right) A \times I_{VC \times HC} \right]$$

**Figure 16. Equation. Predicted FI crashes on horizontal curves and tangents at type 1 crest vertical curves (general form).**

$$N_{PDO} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 \left( \frac{5,730}{R} \right) A \times I_{VC \times HC} \right]$$

**Figure 17. Equation. Predicted PDO crashes on horizontal curves and tangents at type 1 crest vertical curves (general form).**

Where:

A = abs( $G_1 - G_2$ ) (percent); not applicable for level tangents.

$G_1$  = Initial grade (percent) (positive for upgrade; negative for downgrade).

$G_2$  = Final grade (percent) (positive for upgrade; negative for downgrade).

$I_{VC \times HC}$  = Combined vertical and horizontal curve indicator (1 for combined vertical and horizontal curves; 0 otherwise).

The regression results, including the significant interaction, are shown in table 10.

**Table 10. FI and PDO crash modeling results for horizontal curves and tangents at type 1 crest vertical curves.**

Parameter Description	Regression Coefficient	Coefficient Estimate	Standard Error	Lower 95 Percent Confidence Limit	Upper 95 Percent Confidence Limit	Chi-Squared Statistic	Significance Level
<b>FI Crashes/Mi/Year</b>							
Intercept	$b_0$	-9.56	0.23	-10.01	-9.11	N/A	N/A
ln(AADT)	$b_1$	1.09	0.03	1.04	1.15	1,661.0	< 0.0001
1/R × A interaction <sup>a</sup>	$b_2$	0.0088	0.003	0.004	0.014	11.1	0.001
Dispersion	N/A	0.70	0.05	0.60	0.81	N/A	N/A
<b>PDO Crashes/Mi/Year</b>							
Intercept	$b_0$	-8.46	0.20	-8.85	-8.08	N/A	N/A
ln(AADT)	$b_1$	1.01	0.02	0.96	1.05	1,858.8	< 0.0001
1/R × A interaction <sup>a</sup>	$b_2$	0.0046	0.002	0.001	0.008	6.4	0.011
Dispersion	N/A	0.72	0.04	0.64	0.82	N/A	N/A

<sup>a</sup>1/R × A interaction = (5,730/R) × A.

N/A = Not applicable.

Using figure 8, the percentage of roadway segments with extremely high observed FI crash frequencies was 6.50 percent across all roadway segments, which was slightly above the expected 5 percent. The percentages of roadway segments with extremely high observed FI crash frequencies were 6.37 percent for level tangents, 6.52 percent for tangents at type 1 crests, and 7.08 percent for horizontal curves on type 1 crests, indicating that a few roadway segments in these roadway categories experienced unusually high FI crash frequencies given the prediction model used. None of the segments experienced extremely low FI crash frequencies under the assumed model at the 5 percent significance level.

Similarly, the percentage of roadway segments with extremely high observed PDO crash frequencies was 6.70 percent across all roadway segments, similar to that for FI crashes. The percentages were 7.00 percent for level tangents, 5.94 percent for tangents at type 1 crests, and 6.59 percent for horizontal curves on type 1 crests, indicating that a few roadway segments in these roadway categories experienced unusually high FI crash frequencies given the prediction model used. None of the segments experienced extremely low FI crash frequencies under the assumed model at the 5 percent significance level.

The average probability of predicting a more extreme than observed FI crash frequency was calculated using figure 7. Across all roadway segments, the average probability was 0.13. The probability was 0.13 for level tangents, 0.11 for tangents at type 1 crests, and 0.11 for horizontal curves on type 1 crests, which are all considerably lower than the theoretically expected value of 0.25.

Similarly, the average probability of predicting a more extreme than observed PDO crash frequency was 0.15 across all roadway segments. The probability was 0.16 for level tangents, 0.14 for tangents on type 1 crests, and 0.13 for horizontal curves on type 1 crests, which are all

considerably lower than the theoretically expected value of 0.25. For both FI and PDO crashes, these low probabilities indicate that the model might not provide an adequate fit to the data.

Substituting the regression coefficients in figure 16 and figure 17 with their corresponding estimates shown in table 10, the prediction models for FI and PDO crashes/mi/year are obtained as shown in figure 18 and figure 19\*:

$$N_{FI} = \exp \left[ -9.56 + 1.09 \ln(\text{AADT}) + 0.0088 \left( \frac{5,730}{R} \right) A \times I_{VC \times HC} \right]$$

**Figure 18. Equation. Predicted FI crashes on horizontal curves and tangents at type 1 crest vertical curves (explicit form).**

$$N_{PDO} = \exp \left[ -8.46 + 1.01 \ln(\text{AADT}) + 0.0046 \left( \frac{5,730}{R} \right) A \times I_{VC \times HC} \right]$$

**Figure 19. Equation. Predicted PDO crashes on horizontal curves and tangents at type 1 crest vertical curves (explicit form).**

For horizontal curves and tangents at type 1 crest vertical curves, only the interaction term between curve radius (a characteristic of horizontal curves) and  $A$ , the algebraic difference in initial and final grades (a vertical curve characteristic), was found to be statistically significant. Crash frequency increased with the combination of decreasing curve radius and an increasing difference in grades.

Of interest is the fact that the marginal effect of the interaction, once the effect of AADT is accounted for, is very small as indicated by the extremely small chi-squared value in comparison to that for AADT (shown in table 10). After traffic volume was accounted for, there was little difference among the three roadway groups (i.e., level tangents, tangents on nonlevel grades, and horizontal curves at type 1 crest vertical curves). The argument can be made that the crash variability in all three groups was large (as seen in table 9). Additionally, the horizontal curves group consisted of only 87 mi of roadway as compared to the 833 mi of level tangents and 200 mi of tangents at type 1 crests.

In summary, the predictive model developed for type 1 crest vertical curves includes a term representing the interaction between the horizontal curve radius and the algebraic difference in grade for the crest vertical curve. The main effects of horizontal curve radius and algebraic difference in grade were not statistically significant. The interaction term indicates that the crash frequency increased as the horizontal curve radius decreased, the length of the vertical curve increased, and the sharpness of the vertical curve increased (i.e., as  $K$  decreases). This interaction term, while not nearly as strongly related to crash frequency as AADT, is potentially useful in the AASHTO HSM because it currently does not contain CMFs for crest vertical curves.<sup>(1)</sup>

A model with a main effect or interaction for  $K$ , representing the sharpness of the vertical curve, would potentially be more useful to designers than a model containing the algebraic difference in grade. However, neither the main effect of  $K$  nor any interactions involving  $K$  were statistically significant for type 1 crest vertical curves. The models in figure 18 and figure 19 can also be written with  $L_{VC}/K$  substituted for  $A$ . The effect expressed in this form makes engineering sense

as it indicates that crash frequency would increase as the length of the vertical curve increases and as the sharpness of the vertical curve increases (i.e., as  $K$  gets smaller). Therefore, it is suggested that figure 18 and figure 19 be recast as **shown in figure 20 and figure 21\***:

$$N_{FI} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 \left( \frac{5,730}{R} \right) \left( \frac{L_{VC}}{K} \right) \times I_{VC \times HC} \right]$$

**Figure 20. Equation. Predicted FI crashes on horizontal curves and tangents at type 1 crest vertical curves (alternate form).**

$$N_{PDO} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 \left( \frac{5,730}{R} \right) \left( \frac{L_{VC}}{K} \right) \times I_{VC \times HC} \right]$$

**Figure 21. Equation. Predicted PDO crashes on horizontal curves and tangents at type 1 crest vertical curves (alternate form).**

### Horizontal Curves and Tangents at Type 1 Sag Vertical Curves

The following three alignment combinations shaded in figure 22 were included in the analysis:

- Horizontal curves at type 1 sag vertical curves ( $N = 982$  and total roadway length = 57 mi).
- Tangents at type 1 sag vertical curves ( $N = 1,973$  and total roadway length = 145 mi).
- Level tangents—base condition (grade < 1 percent) ( $N = 5,744$  and total roadway length = 833 mi).

	Straight Grades		Vertical Curves			
	Level Abs(Grade) <1 percent	Nonlevel Abs(Grade) ≥ 1 percent	Type 1 Crest	Type 1 Sag	Type 2 Crest	Type 2 Sag
Tangents	<b>Base condition</b>					
Horizontal Curves						

Note: Data in the three shaded cells were used in this analysis, while data in the blank cells were not used in this analysis.

**Figure 22. Illustration. Alignment combinations used in the analysis of horizontal curves and tangents at type 1 sag vertical curves.**

Basic descriptive statistics such as sample size (i.e., number of roadway sections); total roadway length; and minimum, maximum, mean, and median values for specific parameters are shown in table 11 for each of the three alignment types included in the analysis.

**Table 11. Descriptive statistics for horizontal curves and tangents at type 1 sag vertical curves.**

Parameter	Minimum	Maximum	Mean	Median
<b>Horizontal Curves at Type 1 Sag Vertical Curves</b>				
<b>(N=982; total roadway length = 57 mi)</b>				
AADT (vehicles/day)	169	19,373	3,074	1,821
Section length (mi)	0.01	0.31	0.06	0.05
Horizontal curve length (mi)	0.01	1.00	0.15	0.12
Curve radius (ft)	100	11,459	2,085	1,433
Vertical curve length (ft)	92	2,200	545	500
A	1.0	13.0	4.4	3.8
K	10.4	966.2	153.2	116.4
FI crashes per MVMT	0	36.61	0.71	0
PDO crashes per MVMT	0	21.35	0.81	0
Total crashes per MVMT	0	52.06	1.53	0
<b>Tangents at Type 1 Sag Vertical Curves</b>				
<b>(N=1,973; total roadway length = 145 mi)</b>				
AADT (vehicles/day)	175	26,088	3,098	1,828
Section length (mi)	0.01	0.51	0.07	0.06
Horizontal curve length (mi)				
Curve radius (ft)				
Vertical curve length (ft)	60	2,800	523	400
A	1.0	15.1	4.2	3.6
K	6.8	969.7	153.0	120.2
FI crashes per MVMT	0	46.26	0.48	0
PDO crashes per MVMT	0	40.11	0.65	0
Total crashes per MVMT	0	70.19	1.12	0
<b>Level Tangents—Base Condition</b>				
<b>(N=5,744; total roadway length = 833 mi)</b>				
AADT (vehicles/day)	169	26,088	3,287	2,160
Section length (mi)	0.01	2.10	0.15	0.09
Horizontal curve length (mi)				
Curve radius (ft)				
Vertical curve length (ft)				
A				
K				
FI crashes per MVMT	0	34.21	0.46	0
PDO crashes per MVMT	0	39.50	0.67	0
Total crashes per MVMT	0	55.38	1.14	0

Note: No roadway segments exist in the shaded cells.

The final crash prediction models for FI and PDO crashes are as shown in figure 23 and figure 24\*:

$$N_{FI} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 \frac{1}{K} \times I_{VC} + b_3 \left( \frac{5,730}{R} \right) A \times I_{VC \times HC} \right]$$

**Figure 23. Equation. Predicted FI crashes on horizontal curves and tangents at type 1 sag vertical curves (general form).**

$$N_{PDO} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 \frac{1}{K} \times I_{VC} + b_3 \left( \frac{5,730}{R} \right) A \times I_{VC \times HC} \right]$$

**Figure 24. Equation. Predicted PDO crashes on horizontal curves and tangents at type 1 sag vertical curves (general form).**

Where:

$K = L_{VC}/A$ ; not applicable for level tangents.

$L_{VC}$  = Vertical curve length (ft).

$I_{VC}$  = Vertical curve indicator (1 for vertical curves; 0 otherwise).

The regression results, including all statistically significant parameters and interaction, are shown in table 12.

**Table 12. FI and PDO crash modeling results for horizontal curves and tangents at type 1 sag vertical curves.**

Parameter Description	Regression Coefficient	Coefficient Estimate	Standard Error	Lower 95 percent Confidence Limit	Upper 95 percent Confidence Limit	Chi-Squared Statistic	Significance Level
<b>FI Crashes/Mi/Year</b>							
Intercept	$b_0$	-9.55	0.24	-10.02	-9.08	N/A	N/A
ln(AADT)	$b_1$	1.10	0.03	1.04	1.15	1,516.6	< 0.0001
1/K	$b_2$	10.51	5.18	0.36	20.66	3.9	0.048
1/R × A interaction <sup>a</sup>	$b_3$	0.011	0.003	0.005	0.017	12.3	0.0005
Dispersion	N/A	0.86	0.06	0.75	0.99	N/A	N/A
<b>PDO Crashes/Mi/Year</b>							
Intercept	$b_0$	-8.63	0.20	-9.03	-8.24		
ln(AADT)	$b_1$	1.03	0.03	0.98	1.08	1,776.9	< 0.0001
1/K	$b_2$	8.62	4.41	-0.02	17.26	3.7	0.055
1/R × A interaction <sup>a</sup>	$b_3$	0.010	0.002	0.005	0.014	16.7	< 0.0001
Dispersion	N/A	0.79	0.05	0.70	0.89	N/A	N/A

<sup>a</sup>1/R × A interaction = (5,730/R) × A.

N/A = Not applicable.

Using figure 8, the percentage of roadway segments with extremely high observed FI crash frequencies was 5.90 percent across all roadway segments, slightly above the expected 5 percent. The percentages of roadway segments with extremely high observed FI crash frequencies were

6.03 percent for level tangents, 5.23 percent for tangents at type 1 sags, and 6.51 percent for horizontal curves on type 1 sags, indicating that a few roadway segments in these roadway categories experienced unusually high FI crash frequencies given the prediction model used. None of the segments experienced extremely low FI crash frequencies under the assumed model at the 5 percent significance level.

Similarly, the percentage of roadway segments with extremely high observed PDO crash frequencies was 6.60 percent across all roadway segments, slightly higher than that for FI crashes. The percentages were 6.79 percent for level tangents, 6.39 percent for tangents at type 1 sags, and 5.90 percent for horizontal curves on type 1 sags, indicating that a few roadway segments in these roadway categories experienced unusually high FI crash frequencies given the prediction model used. None of the segments experienced extremely low FI crash frequencies under the assumed model at the 5 percent significance level.

The average probability of predicting a more extreme than observed FI crash frequency was calculated using figure 7. Across all roadway segments, the average probability was 0.13. The probability was 0.14 for level tangents, 0.11 for tangents at type 1 sags, and 0.12 for horizontal curves on type 1 sags, which were all considerably lower than the theoretically expected value of 0.25.

Similarly, the average probability of predicting a more extreme than observed PDO crash frequency was 0.15 across all roadway segments. The probability was 0.16 for level tangents, 0.14 for tangents on type 1 sags, and 0.14 for horizontal curves on type 1 sags, which were all considerably lower than the theoretically expected value of 0.25. For both FI and PDO crashes, these low probabilities indicate that the model might not provide an adequate fit to the data.

Substituting the regression coefficients in figure 23 and figure 24 with their corresponding estimates in table 12, the prediction models for FI and PDO crashes/mi/year are as **shown in figure 25 and figure 26\***:

$$N_{FI} = \exp \left[ -9.55 + 1.10 \ln(\text{AADT}) + 10.51 \frac{1}{K} \times I_{VC} + 0.011 \left( \frac{5,730}{R} \right) A \times I_{VC \times HC} \right]$$

**Figure 25. Equation. Predicted FI crashes on horizontal curves and tangents at type 1 sag vertical curves (explicit form).**

$$N_{PDO} = \exp \left[ -8.63 + 1.03 \ln(\text{AADT}) + 8.62 \frac{1}{K} \times I_{VC} + 0.010 \left( \frac{5,730}{R} \right) A \times I_{VC \times HC} \right]$$

**Figure 26. Equation. Predicted PDO crashes on horizontal curves and tangents at type 1 sag vertical curves (explicit form).**

For horizontal curves and tangents at type 1 sag vertical curves, the inverse of  $K$ , a measure of vertical curvature, and the interaction term between curve radius (a characteristic of horizontal curves) and  $A$  (a vertical curve characteristic) were found to be statistically significant in both models. These models show that crash frequency increases with decreasing  $K$  (i.e., with sharper sags). The interaction term between  $R$  and  $A$  shows that crash frequency increases with the decreasing curve radius and increasing difference in grades. In both models, the effect of the interaction is greater than that of  $K$  as reflected in the comparison of their chi-squared values in table 12.

As noted earlier for the crash model at type 1 crest vertical curves shown in figure 18, figure 19, and table 10, the marginal effect of the parameters in each model after the effect of AADT is accounted for is small, as indicated by the small chi-squared values in comparison to that for AADT (see table 12). This is an indication that after traffic volume is accounted for, there is little difference among the three groups of roadway (i.e., level tangents, tangents on nonlevel grades, and horizontal curves at type 1 sag vertical curves). The argument can be made that the crash variability in all three groups is large (see in table 11). Additionally, the horizontal curves group consists of only 57 mi of roadway as compared to the 833 mi of level tangents and 145 mi of tangents at type 1 sags.

As in the case for type 1 crest vertical curves, it makes engineering sense to replace  $A$  with  $L_{VC}/K$  in figure 25 and figure 26. In this form, both the main effect and the interaction term include  $K$  in the denominator rather than having an inconsistency with the main effect including  $K$  and the interaction including  $A$ . Thus, it is recommended that figure 25 and figure 26 be recast as shown in figure 27 and figure 28\*:

$$N_{FI} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 \frac{1}{K} \times I_{VC} + b_3 \left( \frac{5,730}{R} \right) \left( \frac{L_{VC}}{K} \right) \times I_{VC \times HC} \right]$$

**Figure 27. Equation. Predicted FI crashes on horizontal curves and tangents at type 1 sag vertical curves (alternate form).**

$$N_{PDO} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 \frac{1}{K} \times I_{VC} + b_3 \left( \frac{5,730}{R} \right) \left( \frac{L_{VC}}{K} \right) \times I_{VC \times HC} \right]$$

**Figure 28. Equation. Predicted PDO crashes on horizontal curves and tangents at type 1 sag vertical curves (alternate form).**

In summary, the predictive model developed for type 1 sag vertical curves includes a term representing the sharpness of the vertical curve as a main effect and an interaction between the horizontal curve radius, the vertical curve length, and the sharpness of the vertical curve. The model indicates that crash frequency increases as the horizontal curve radius decreases, the length of the vertical curve increases, and the sharpness of the vertical curve increases (i.e., as  $K$  decreases). The main effect and the interaction, while not as strongly related to crash frequency as AADT, are potentially useful in the AASHTO HSM because it does not currently contain CMFs for sag vertical curves.<sup>(1)</sup>

### Horizontal Curves and Tangents at Type 2 Crest Vertical Curves

The following three alignment combinations shaded in figure 29 were included in the analysis:

- Horizontal curves at type 2 crest vertical curves ( $N = 1,071$  and total roadway length = 62 mi).
- Tangents at type 2 crest vertical curves ( $N = 1,922$  and total roadway length = 132 mi).
- Level tangents—base condition (grade < 1 percent) ( $N = 5,742$  and total roadway length = 833 mi).

	Straight Grades		Vertical Curves			
	Level Abs(Grade) <1 percent	Nonlevel Abs(Grade) ≥ 1 percent	Type 1 Crest	Type 1 Sag	Type 2 Crest	Type 2 Sag
Tangents	<b>Base condition</b>					
Horizontal Curves						

Note: Data in the three shaded cells were used in this analysis, while data in the blank cells were not used in this analysis.

**Figure 29. Illustration. Alignment combinations used in the analysis of horizontal curves and tangents at type 2 crest vertical curves.**

Basic descriptive statistics such as sample size (i.e., number of roadway sections); total roadway length; and minimum, maximum, mean, and median values for specific parameters are shown in table 13 for each of the three alignment types included in the analysis.

**Table 13. Descriptive statistics for horizontal curves and tangents at type 2 crest vertical curves.**

Parameter	Minimum	Maximum	Mean	Median
<b>Horizontal Curves at Type 2 Crest Vertical Curves (N=1,071; total roadway length = 62 mi)</b>				
AADT (vehicles/day)	202	20,931	2,603	1,607
Section length (mi)	0.01	0.34	0.06	0.05
Horizontal curve length (mi)	0.01	1.09	0.16	0.12
Curve radius (ft)	100	11,459	1,960	1,433
Vertical curve length (ft)	75	2,400	543	400
A	1.0	8.3	2.8	2.4
K	15.9	952.4	227.0	178.8
FI crashes per MVMT	0	28.16	0.63	0
PDO crashes per MVMT	0	30.02	0.87	0
Total crashes per MVMT	0	30.02	1.50	0
<b>Tangents at Type 2 Crest Vertical Curves (N=1,922; total roadway length = 132 mi)</b>				
AADT (vehicles/day)	175	21,825	2,741	1,652
Section length (mi)	0.01	0.38	0.07	0.06
Horizontal curve length (mi)				
Curve radius (ft)				
Vertical curve length (ft)	60	2,400	498	400
A	1.0	8.0	2.6	2.2
K	16.2	985.9	222.4	176.3
FI crashes per MVMT	0	36.12	0.42	0
PDO crashes per MVMT	0	27.05	0.61	0

Parameter	Minimum	Maximum	Mean	Median
Total crashes per MVMT	0	36.12	1.03	0
<b>Level Tangents—Base Condition</b> (N=5,742; total roadway length = 833 mi)				
AADT (vehicles/day)	169	26,088	3,287	2,160
Section length (mi)	0.01	2.10	0.15	0.09
Horizontal curve length (mi)				
Curve radius (ft)				
Vertical curve length (ft)				
A				
K				
FI crashes per MVMT	0	34.21	0.46	0
PDO crashes per MVMT	0	31.60	0.66	0
Total crashes per MVMT	0	36.34	1.12	0

Note: No roadway segments exist in the shaded cells.

The final crash prediction models for FI and PDO crashes are as shown in figure 30 and figure 31\*:

$$N_{FI} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 \ln \left( 2 \times \frac{5,730}{R} \right) \times I_{HC} \right]$$

**Figure 30. Equation. Predicted FI crashes on horizontal curves and tangents at type 2 crest vertical curves (general form).**

$$N_{PDO} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 \ln \left( 2 \times \frac{5,730}{R} \right) \times I_{HC} \right]$$

**Figure 31. Equation. Predicted PDO crashes on horizontal curves and tangents at type 2 crest vertical curves (general form).**

The regression results, including all statistically significant parameters, are shown in table 14. There were no statistically significant interactions for type 2 crest vertical curves.

Using figure 8, the percentage of roadway segments with extremely high observed FI crash frequencies was 6.46 percent across all roadway segments, slightly above the expected percentage, which was 5 percent. The percentages of roadway segments with extremely high observed FI crash frequencies were 6.46 percent for level tangents, 5.84 percent for tangents at type 2 crests, and 7.55 percent for horizontal curves on type 2 crests, indicating that a few roadway segments in these roadway categories experienced unusually high FI crash frequencies given the prediction model used. None of the segments experienced extremely low FI crash frequencies under the assumed model at the 5 percent significance level.

**Table 14. FI and PDO crash modeling results for horizontal curves and tangents at type 2 crest vertical curves.**

Parameter Description	Regression Coefficient	Coefficient Estimate	Standard Error	Lower 95 percent Confidence Limit	Upper 95 percent Confidence Limit	Chi-Squared Statistic	Significance Level
<b>FI Crashes/Mi/Year</b>							
Intercept	$b_0$	-9.52	0.24	-9.99	-9.05	N/A	N/A
ln(AADT)	$b_1$	1.09	0.03	1.03	1.14	1,470.5	< 0.0001
1/R term <sup>a</sup>	$b_2$	0.20	0.04	0.12	0.28	20.4	< 0.0001
Dispersion	N/A	0.67	0.06	0.57	0.79	N/A	N/A
<b>PDO Crashes/Mi/Year</b>							
Intercept	$b_0$	-8.38	0.20	-8.78	-7.99	N/A	N/A
ln(AADT)	$b_1$	1.00	0.02	0.95	1.05	1,699.0	< 0.0001
1/R term <sup>a</sup>	$b_2$	0.10	0.04	0.03	0.18	6.6	0.010
Dispersion	N/A	0.65	0.05	0.57	0.74	N/A	N/A

<sup>a</sup>1/R term =  $\ln(2 \times 5,730/R)$ .

N/A = Not applicable.

The percentage of roadway segments with extremely high observed PDO crash frequencies was 6.93 percent across all roadway segments, slightly higher than that for FI crashes. The percentages were 6.99 percent for level tangents, 6.61 percent for tangents at type 2 crests, and 7.16 percent for horizontal curves on type 2 crests, indicating that a few roadway segments in these roadway categories experienced unusually high FI crash frequencies given the prediction model used. None of the segments experienced extremely low FI crash frequencies under the assumed model at the 5 percent significance level.

The average probability of predicting a more extreme than observed FI crash frequency was calculated using figure 7. Across all roadway segments, the average probability was 0.12. The probability was 0.13 for level tangents, 0.09 for tangents at type 2 crests, and 0.11 for horizontal curves on type 2 crests, which were all considerably lower than the theoretically expected value of 0.25.

Similarly, the average probability of predicting a more extreme than observed PDO crash frequency was 0.14 across all roadway segments. The probability was 0.16 for level tangents, 0.12 for tangents on type 2 crests, and 0.12 for horizontal curves on type 2 crests, which were all considerably lower than the theoretically expected value of 0.25. For both FI and PDO crashes, these low probabilities indicate that the model might not provide an adequate fit to the data.

Substituting the regression coefficients in figure 30 and figure 31 with their corresponding estimates in table 14, the prediction models for FI and PDO crashes/mi/year are as shown in figure 32 and figure 33\*:

$$N_{FI} = \exp \left[ -9.52 + 1.09 \ln(\text{AADT}) + 0.20 \ln \left( 2 \times \frac{5,730}{R} \right) \times I_{HC} \right]$$

**Figure 32. Equation. Predicted FI crashes on horizontal curves and tangents at type 2 crest vertical curves (explicit form).**

$$N_{PDO} = \exp \left[ -8.38 + 1.00 \ln(\text{AADT}) + 0.10 \ln \left( 2 \times \frac{5,730}{R} \right) \times I_{HC} \right]$$

**Figure 33. Equation. Predicted PDO crashes on horizontal curves and tangents at type 2 crest vertical curves (explicit form).**

The horizontal curves group consists of only 62 mi of roadway as compared to the 833 mi of level tangents and 132 mi of tangents at type 2 crests. For horizontal curves and tangents at type 2 crest vertical curves, curve radius (a characteristic of horizontal curves) was the only parameter found to be statistically significant in both models. Although its effect is more pronounced for FI than for PDO crashes, it is small compared to that of AADT. It is interesting to note that the effect of curve radius for horizontal curves and tangents at type 2 crest vertical curves is comparable to the effect of curve radius for horizontal curves on straight grades (compare coefficient estimates in table 14 to those in table 8).

The models developed for type 2 crest vertical curves include a main effect for horizontal curve radius but no effect for any vertical curve elements. This makes the models for type 2 crest vertical curves less useful than the models for type 1 crest vertical curves. Sensitivity analyses are needed to establish whether the straight grade models in figure 10 and figure 11 might be applied to type 2 crest vertical curves, with  $G$  set equal to the average of  $G_1$  and  $G_2$  instead of figure 32 and figure 33.

### Horizontal Curves and Tangents at Type 2 Sag Vertical Curves

The following three alignment combinations shaded in figure 34 were included in the analysis:

- Horizontal curves at type 2 sag vertical curves ( $N = 1,217$  and total roadway length = 63 mi).
- Tangents at type 2 sag vertical curves ( $N = 2,174$  and total roadway length = 129 mi).
- Level tangents—base condition (grade < 1 percent) ( $N = 5,741$  and total roadway length = 833 mi).

	Straight Grades		Vertical Curves			
	Level Abs(Grade) <1 percent	Nonlevel Abs(Grade) ≥ 1 percent	Type 1 Crest	Type 1 Sag	Type 2 Crest	Type 2 Sag
Tangents	<b>Base condition</b>					
Horizontal Curves						

Note: Data in the three shaded cells were used in this analysis, while data in the blank cells were not used in this analysis.

**Figure 34. Illustration. Alignment combinations used in the analysis of horizontal curves and tangents at type 2 sag vertical curves.**

Basic descriptive statistics such as sample size (i.e., number of roadway sections); total roadway length; and minimum, maximum, mean, and median values for specific parameters are shown in table 15 for each of the three alignment types included in the analysis.

**Table 15. Descriptive statistics for horizontal curves and tangents at type 2 sag vertical curves.**

Parameter	Minimum	Maximum	Mean	Median
<b>Horizontal Curves at Type 2 Sag Vertical Curves (N=1,217; total roadway length = 63 mi)</b>				
AADT (vehicles/day)	175	21,825	2,691	1,742
Section length (mi)	0.01	0.30	0.05	0.04
Horizontal curve length (mi)	0.01	1.09	0.17	0.13
Curve radius (ft)	100	11,459	1,964	1,433
Vertical curve length (ft)	60	1,600	424	400
A	1.0	7.7	2.7	2.5
K	9.7	917.4	185.6	149.3
FI crashes per MVMT	0	26.06	0.60	0
PDO crashes per MVMT	0	27.82	0.95	0
Total crashes per MVMT	0	27.82	1.54	0
<b>Tangents at Type 2 Sag Vertical Curves (N=2,174; total roadway length = 129 mi)</b>				
AADT (vehicles/day)	169	23,334	2,909	1,776
Section length (mi)	0.01	0.38	0.06	0.05
Horizontal curve length (mi)				
Curve radius (ft)				
Vertical curve length (ft)	60	2,000	400	400
A	1.0	7.6	2.6	2.2
K	16.2	970.9	184.3	148.1
FI crashes per MVMT	0	27.82	0.44	0
PDO crashes per MVMT	0	28.27	0.61	0
Total crashes per MVMT	0	38.20	1.05	0
<b>Level Tangents—Base Condition (N=5,741; total roadway length = 833 mi)</b>				
AADT (vehicles/day)	169	26,088	3,288	2,160
Section length (mi)	0.01	2.10	0.15	0.09
Horizontal curve length (mi)				
Curve radius (ft)				
Vertical curve length (ft)				
A				
K				
FI crashes per MVMT	0	34.21	0.46	0
PDO crashes per MVMT	0	25.79	0.66	0
Total crashes per MVMT	0	36.34	1.12	0

Note: No roadway segments exist in the shaded cells.

The final crash prediction models for FI and PDO crashes are shown in figure 35 and figure 36\*:

$$N_{FI} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 \ln \left( 2 \times \frac{5,730}{R} \right) \times I_{HC} \right]$$

**Figure 35. Equation. Predicted FI crashes on horizontal curves and tangents at type 2 sag vertical curves (general form).**

$$N_{PDO} = \exp \left[ b_0 + b_1 \ln(\text{AADT}) + b_2 \left( \frac{5,730}{R} \right) A \times I_{VC \times HC} \right]$$

**Figure 36. Equation. Predicted PDO crashes on horizontal curves and tangents at type 2 sag vertical curves (general form).**

The regression results, including all statistically significant parameters and interaction, are shown in table 16.

**Table 16. FI and PDO crash modeling results for horizontal curves and tangents at type 2 sag vertical curves.**

Parameter Description	Regression Coefficient	Coefficient Estimate	Standard Error	Lower 95 percent Confidence Limit	Upper 95 percent Confidence Limit	Chi-Squared Statistic	Significance Level
<b>FI Crashes/Mi/Year</b>							
Intercept	$b_0$	-9.42	0.24	-9.90	-8.95	N/A	N/A
ln(AADT)	$b_1$	1.08	0.03	1.02	1.13	1,427.2	< 0.0001
1/R term <sup>a</sup>	$b_2$	0.188	0.04	0.11	0.27	18.2	< 0.0001
Dispersion	N/A	0.76	0.06	0.65	0.88	NA	NA
<b>PDO Crashes/Mi/Year</b>							
Intercept	$b_0$	-8.30	0.20	-8.69	-7.90	N/A	N/A
ln(AADT)	$b_1$	0.99	0.02	0.94	1.03	1,648.2	< 0.0001
1/R × A interaction <sup>b</sup>	$b_2$	0.022	0.005	0.013	0.031	20.8	< 0.0001
Dispersion	N/A	0.64	0.05	0.56	0.73	N/A	N/A

<sup>a</sup>1/R term =  $\ln(2 \times 5,730/R)$ .

<sup>b</sup>1/R × A interaction =  $(5,730/R) \times A$ .

N/A = Not applicable.

Using figure 8, the percentage of roadway segments with extremely high observed FI crash frequencies was 6.08 percent across all roadway segments, which was slightly above the expected percentage of 5 percent. The percentage of roadway segments with extremely high observed FI crash frequencies was 6.24 percent for level tangents, 5.53 percent for tangents at type 2 sags, and 6.32 percent for horizontal curves on type 2 sags, indicating that a few roadway segments in these categories experienced unusually high FI crash frequencies given the prediction model used. None of the segments experienced extremely low FI crash frequencies under the assumed model at the 5 percent significance level.

Similarly, the percentage of roadway segments with extremely high observed PDO crash frequencies was 6.99 percent across all roadway segments, which was slightly higher than that

for FI crashes. The percentages were 7.10 percent for level tangents, 6.46 percent for tangents at type 2 sags, and 7.42 percent for horizontal curves on type 2 sags, indicating that a few roadway segments in these roadway categories experienced unusually high FI crash frequencies given the prediction model used. None of the segments experienced extremely low FI crash frequencies under the assumed model at the 5 percent significance level.

The average probability of predicting a more extreme than observed FI crash frequency was calculated using figure 7. Across all roadway segments, the average probability was 0.12. The probability was 0.13 for level tangents, 0.09 for tangents at type 2 sags, and 0.10 for horizontal curves on type 2 sags, which were all considerably lower than the theoretically expected value of 0.25.

Similarly, the average probability of predicting a more extreme than observed PDO crash frequency was 0.14 across all roadway segments. The probability was 0.16 for level tangents, 0.12 for tangents on type 2 sags, and 0.12 for horizontal curves on type 2 sags, which were all considerably lower than the theoretically expected value of 0.25. For both FI and PDO crashes, these low probabilities indicate that the model might not provide an adequate fit to the data.

Substituting the regression coefficients in figure 35 and figure 36 with their corresponding estimates in table 16, the prediction models for FI and PDO crashes/mi/year are as shown in figure 37 and figure 38\*:

$$N_{FI} = \exp \left[ -9.42 + 1.08 \ln(\text{AADT}) + 0.188 \ln \left( 2 \times \frac{5,730}{R} \right) \times I_{HC} \right]$$

**Figure 37. Equation. Predicted FI crashes on horizontal curves and tangents at type 2 sag vertical curves (explicit form).**

$$N_{PDO} = \exp \left[ -8.30 + 0.99 \ln(\text{AADT}) + 0.022 \left( \frac{5,730}{R} \right) A \times I_{VC \times HC} \right]$$

**Figure 38. Equation. Predicted PDO crashes on horizontal curves and tangents at type 2 sag vertical curves (explicit form).**

The horizontal curves group consists of only 63 mi of roadway as compared to 833 mi of level tangents and 129 mi of tangents at type 2 sags. For horizontal curves and tangents at type 2 sag vertical curves, curve radius (a characteristic of horizontal curves) was the only parameter found to be statistically significant in the FI crash prediction model. The model in figure 37 shows that FI crash frequency increases with decreasing curve radius, as expected. For the PDO crash prediction model, only the interaction term between curve radius (a characteristic of horizontal curves) and A (a vertical curve characteristic), was found to be statistically significant. The model in figure 38 shows that crash frequency increases with the combination of decreasing curve radius and increasing difference in grades.

In either model, the effect of the significant parameter or interaction of parameters is more pronounced than in previous models as reflected in the slightly higher chi-squared values. The magnitude of the coefficients is consistent between this set of models and previous models. There is a substantial inconsistency between the FI and PDO crash models for type 2 sag vertical curves in that the PDO crash model includes a statistically significant interaction between horizontal curve radius and algebraic difference in grade, but the FI crash model does not. This

is the only situation in the study where different parameters or functional forms had to be used for the corresponding FI and PDO crash models. This inconsistency makes the models for type 2 sag vertical curves less useful than the models for type 1 sag vertical curves.  $L_{VC}/K$  could be substituted for  $A$  in figure 38, as was done for type 1 crest vertical curves and type 1 sag vertical curves, but it seems pointless to do this in the PDO crash model when the more important FI crash model does not include a similar interaction term. Sensitivity analyses might establish whether, as an alternative to the inconsistent models in figure 37 and figure 38, the straight grade models in figure 10 and figure 11 might be applied to type 2 sag vertical curves with the value of  $G$  set equal to the average of  $G_1$  and  $G_2$ .

## STUDY LIMITATIONS

The major limitation of the study results is that data from only one State were used in the analysis. It would have been desirable to use data from additional States, but Washington is the only known State for which curve and grade data are available for the entire State highway system in a form that can be linked to crash data.

The study divided the roadway into segments for analysis. A new segment started at each point where a horizontal curve or vertical alignment feature (straight grade or vertical curve) began or ended. Segmenting the roadway in this way resulted in a number of very short segments. The shortest of these segments were dropped from the analysis, but some of the segments that were analyzed may be shorter than the expected accuracy of reported crash locations. Nevertheless, this approach to segmenting roadways appears to be the best method to the authors of this report for analyzing these data because the geometrics of each analysis section are known to be homogeneous with respect to horizontal and vertical alignment features. Alternative analysis approaches that use fixed-length study sections (typically 0.5 or 1.0 mi long) and model the percentage of section length over which particular geometric features (or combinations of features) are present do not appear likely to provide satisfactory results for this application.

As noted earlier, the analysis considered the effects of horizontal and vertical geometrics on crashes but did not consider other features known to affect crashes such as lane and shoulder widths. In fact, the AASHTO HSM includes 12 factors that are known to affect safety on rural two-lane highways.<sup>(1)</sup> Consideration of all these factors was not beyond the scope of this current study. A future study is necessary to develop safety prediction models for rural two-lane highways considering all 12 factors and their interactions.

## CHAPTER 5—CRASH MODIFICATION FACTORS

CMFs used in the AASHTO HSM can be derived from the predictive models developed in chapter 4 of this current report. A CMF is a factor that represents the effect on crash frequency for a given crash severity level of varying geometric design or traffic control feature of interest (or a particular combination of geometric design or traffic control feature). Each CMF has a nominal value of 1.0 for a specified base condition. A CMF with a value greater than 1.0 represents a condition for which more crashes would be expected for the base condition. A CMF with a value less than 1.0 represents a condition for which fewer crashes would be expected than for the base condition. The base condition for all CMFs developed in this research is a level tangent roadway.

For each combination of alignment type (and for FI crashes and PDO crashes), CMFs were calculated as the ratio of the predicted crash frequency for a given horizontal curve and grade combination to the predicted crash frequency for the level tangent base condition. The following subsections provide equations for each CMF and figures illustrating the relationships of each roadway parameter to crash frequency and to the relevant CMF. Sample CMF calculations are also presented.

### HORIZONTAL CURVES AND TANGENTS ON STRAIGHT GRADES

CMFs for horizontal curves and tangents on straight grades can be derived from figure 12 and figure 13 as shown in figure 39 and figure 40\*:

$$CMF_{SG,FI} = \begin{cases} \exp \left[ 0.044G + 0.19 \ln \left( 2 \times \frac{5,730}{R} \right) + 4.52 \left( \frac{1}{R} \right) \left( \frac{1}{L_C} \right) \right] & \text{for horizontal curves} \\ \exp[0.044G] & \text{for tangents on nonlevel grades} \\ 1.0 & \text{for level tangents (base condition)} \end{cases}$$

**Figure 39. Equation. FI CMF for horizontal curves and tangents on straight grades.**

$$CMF_{SG,PDO} = \begin{cases} \exp \left[ 0.040G + 0.13 \ln \left( 2 \times \frac{5,730}{R} \right) + 3.80 \left( \frac{1}{R} \right) \left( \frac{1}{L_C} \right) \right] & \text{for horizontal curves} \\ \exp[0.040G] & \text{for tangents on nonlevel grades} \\ 1.0 & \text{for level tangents (base condition)} \end{cases}$$

**Figure 40. Equation. PDO CMF for horizontal curves and tangents on straight grades.**

The functional relationships shown in figure 12 (crashes/mi/year) and figure 39 (CMF) for FI crashes are illustrated in figure 41 for combinations of horizontal curve lengths and percent grades. Curve radius ranged from 100 to 11,460 ft; AADT was fixed at 2,000 vehicles/day; the median traffic volume was for rural two-lane roadways in the database; the horizontal curve length was set at 0.05, 0.10, 0.15, and 0.20 mi; and grade was set at level (i.e., 0 percent) and 1 to 6 percent in increments of 1 percent. Similarly, figure 42 illustrates the relationships shown in figure 13 and figure 40 for PDO crashes.

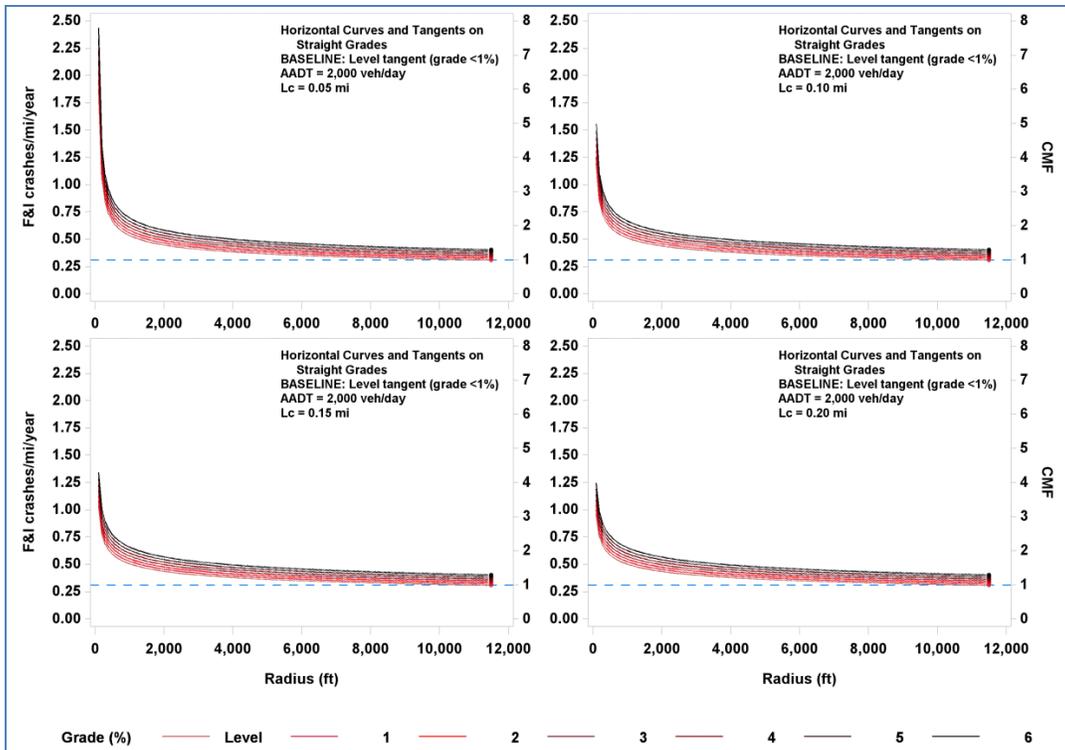


Figure 41. Graph. Predicted FI crashes/mi/year and CMFs for horizontal curves and tangents on straight grades.

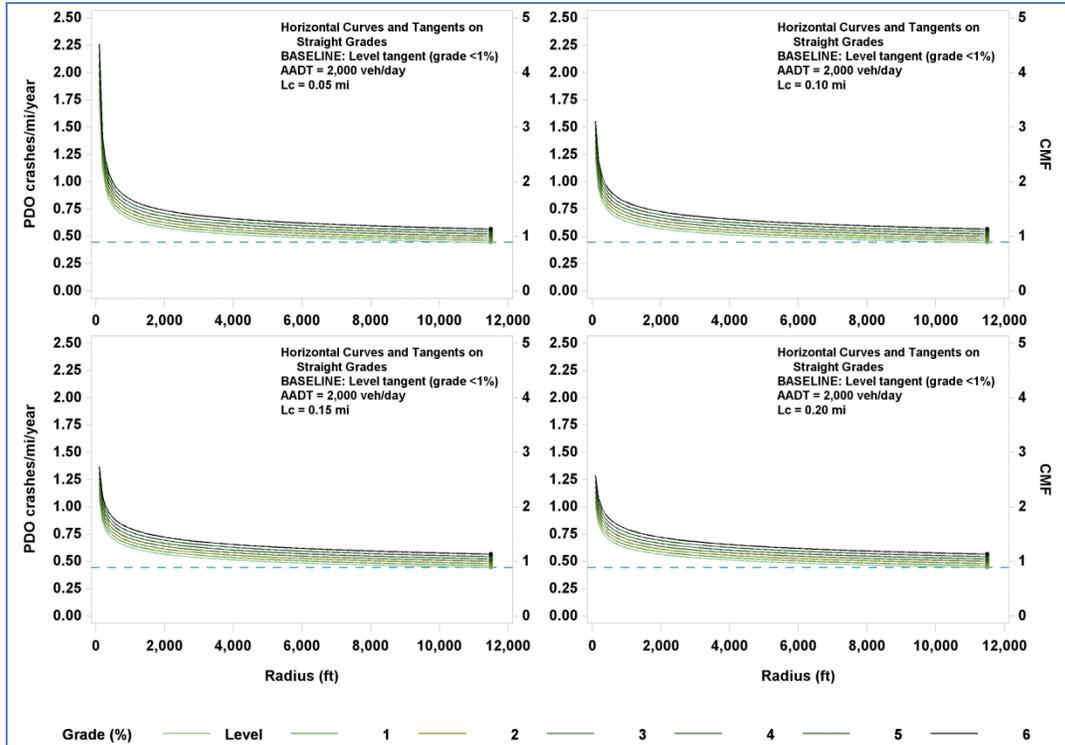


Figure 42. Graph. Predicted PDO crashes/mi/year and CMFs for horizontal curves and tangents on straight grades.

In figure 41 and figure 42, crashes/mi/year are shown on the left y-axis, and the corresponding CMFs are shown on the right y-axis. The dotted blue line corresponds to a base condition tangent with an AADT of 2,000 vehicles/day and therefore has a CMF of 1.0.

To calculate CMF for FI or PDO crashes for a given horizontal curve on a level or nonlevel grade or a tangent on a nonlevel grade,  $G$  (percent),  $R$  (ft), and  $L_C$  (mi) are substituted in figure 39 or figure 40. Example CMFs were calculated for rural two-lane roadways with a  $R$  of 1,433 or 5,730 ft;  $L_C$  of 0.05, 0.10, and 0.50 mi; and  $G$  ranging from level to 6 percent. The results are shown in table 17 for rural two-lane roadways with AADTs from 200 to 26,000 vehicles/day.

**Table 17. Example CMFs for FI and PDO crashes on horizontal curves and tangents on straight grades.**

Grade (Percent)	Tangent on Nonlevel Grade	R = 1,433 ft			R = 5,730 ft		
		Horizontal Curve Length (mi)			Horizontal Curve Length (mi)		
		0.05	0.10	0.50	0.05	0.10	0.50
<b>CMFs for FI Crashes</b>							
Level (< 1 percent)	1.00	1.57	1.53	1.49	1.15	1.15	1.14
1	1.04	1.64	1.59	1.56	1.20	1.20	1.19
2	1.09	1.71	1.67	1.63	1.25	1.25	1.25
3	1.14	1.79	1.74	1.70	1.31	1.31	1.30
4	1.19	1.87	1.82	1.78	1.37	1.36	1.36
5	1.25	1.95	1.9	1.86	1.43	1.42	1.42
6	1.30	2.04	1.98	1.94	1.49	1.49	1.48
<b>CMFs for PDO Crashes</b>							
Level (< 1 percent)	1.00	1.38	1.35	1.32	1.10	1.10	1.10
1	1.04	1.44	1.40	1.38	1.15	1.14	1.14
2	1.08	1.49	1.46	1.43	1.20	1.19	1.19
3	1.13	1.56	1.52	1.49	1.25	1.24	1.24
4	1.17	1.62	1.58	1.55	1.30	1.29	1.29
5	1.22	1.69	1.65	1.62	1.35	1.35	1.34
6	1.27	1.76	1.72	1.69	1.41	1.40	1.40

In any given column in table 17, the CMFs show the increasing effect on crashes of an increasing grade for a horizontal curve or tangent on a straight grade. In any given row for a given curve radius, the CMFs show the increasing effect on crashes of horizontal curve length. The decreasing effect on crashes of longer curve radii is shown by comparing any two columns for the same curve length between the two radii. The effect of a short and sharp curve (due to the last term in figure 39 and figure 40) is reflected in the high CMFs in the third column of the table. As the curve radius increases and becomes a tangent for all practical purposes, the CMFs for the curve approach in value those of a tangent with the same percent grade. When percent grade nears zero (level roadway) and the radius becomes infinite (tangent roadway), the roadway becomes a level tangent, and CMF becomes 1.

## HORIZONTAL CURVES AND TANGENTS AT TYPE 1 CREST VERTICAL CURVES

CMFs for horizontal curves and tangents at type 1 crest vertical curves can be derived from figure 18 and figure 19 as shown in figure 43 and figure 44\*:

$$CMF_{CI,FI} = \left\{ \begin{array}{ll} \exp \left[ 0.0088 \left( \frac{5,730}{R} \right) \frac{L_{VC}}{K} \right] & \text{for horizontal curves} \\ 1.0 & \text{for tangents at type 1 crests} \\ 1.0 & \text{for level tangents (base condition)} \end{array} \right\}$$

**Figure 43. Equation. FI CMF for horizontal curves and tangents at type 1 crest vertical curves.**

$$CMF_{CI,PDO} = \left\{ \begin{array}{ll} \exp \left[ 0.0046 \left( \frac{5,730}{R} \right) \frac{L_{VC}}{K} \right] & \text{for horizontal curves} \\ 1.0 & \text{for tangents at type 1 crests} \\ 1.0 & \text{for level tangents (base condition)} \end{array} \right\}$$

**Figure 44. Equation. PDO CMF for horizontal curves and tangents at type 1 crest vertical curves.**

The functional relationships shown in figure 18 (crashes/mi/year) and figure 43 (CMF) for FI crashes are illustrated in figure 45 for selected values of  $K$ . Curve radius ranged from 100 to 11,460 ft, AADT was fixed at 2,000 vehicles/day, and  $L_{VC}$  was fixed at 500 ft, the median  $L_{VC}$  in the database. Values of  $K$  were set at 250, 125, 83, 63, and 50, which correspond to a grade difference of 2, 4, 6, 8, and 10 percent, respectively, for a curve length of 500 ft. Similarly, figure 46 illustrates the relationships shown in figure 19 and figure 44 for PDO crashes.

To calculate CMF for FI or PDO crashes for a given horizontal curve at a type 1 crest vertical curve, the actual values of  $R$  (ft),  $L_{VC}$  (ft), and parameter  $K$  (ft/percent) are substituted in figure 43 or figure 44. Example CMFs were calculated for rural two-lane roadways with an  $L_{VC}$  of 500 ft,  $R$  of 1,433 or 5,730 ft, and  $K$  values of 250, 125, 83, 63, and 50 ft/percent using figure 43 and figure 44. The results are shown in table 18 for rural two-lane roadways with AADTs from 200 to 26,000 vehicles/day and  $L_{VC}$  of 500 ft.

**Table 18. Example CMFs for FI and PDO crashes on horizontal curves and tangents at type 1 crest vertical curves.**

$K$	Tangent at Crest	FI CMFs		PDO CMFs	
		$R = 1,433$ ft	$R = 5,730$ ft	$R = 1,433$ ft	$R = 5,730$ ft
250	1.0	1.07	1.02	1.04	1.01
125	1.0	1.15	1.04	1.08	1.02
83	1.0	1.24	1.05	1.12	1.03
63	1.0	1.33	1.07	1.16	1.04
50	1.0	1.42	1.09	1.20	1.05

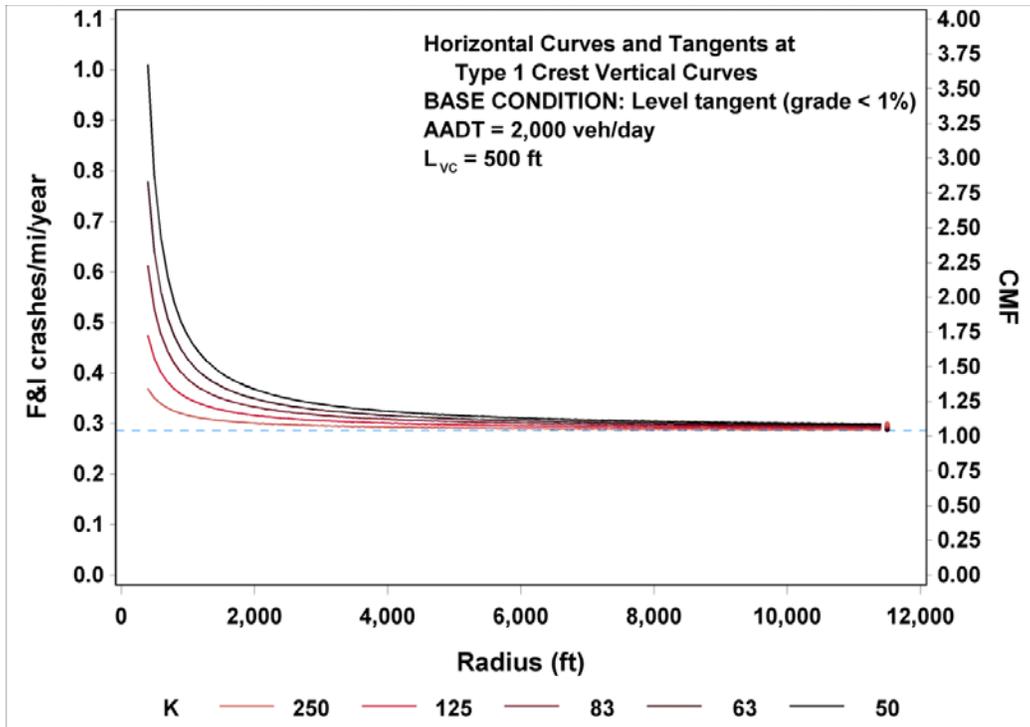


Figure 45. Graph. Predicted FI crashes/mi/year and CMFs for horizontal curves and tangents at type 1 crest vertical curves.

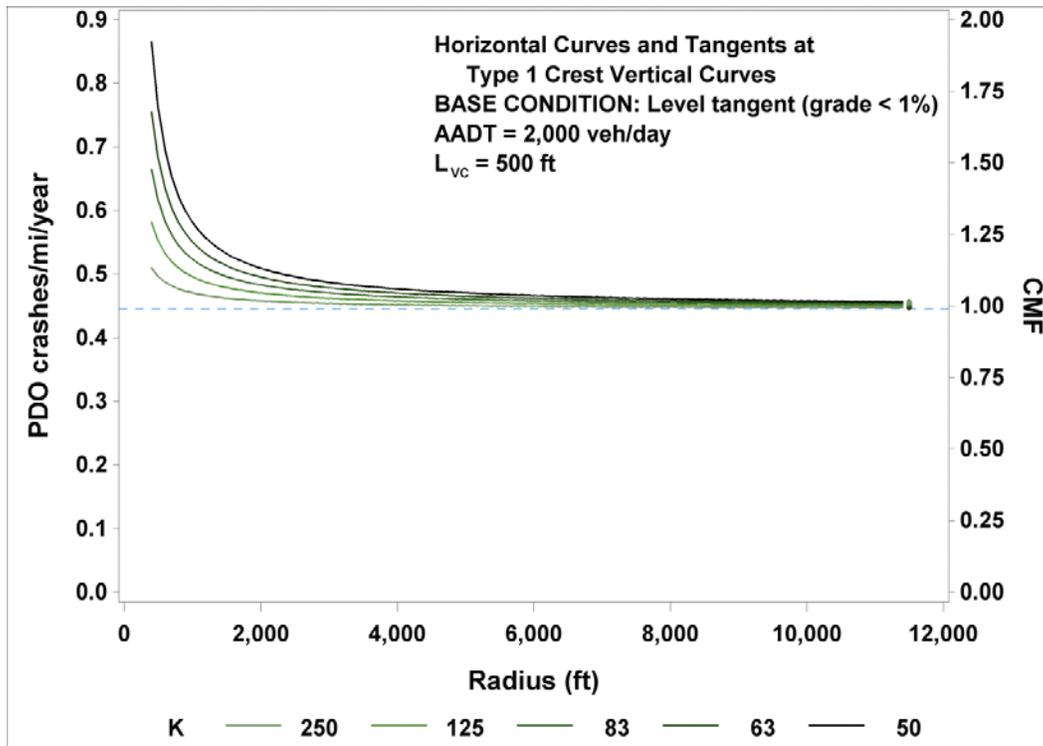


Figure 46. Graph. Predicted PDO crashes/mi/year and CMFs for horizontal curves and tangents at type 1 crest vertical curves.

In any given column in table 18, the CMFs show the increasing effect on crashes of a decreasing  $K$  (steeper crest) of a horizontal curve at a type 1 crest vertical curve. This effect is less pronounced for PDO crashes than for FI crashes. The combined effect of a sharp horizontal curve on a steep vertical crest is reflected in the last rows of the third and fifth columns. When  $K$  becomes infinite (i.e., level roadway) and  $R$  becomes infinite (i.e., tangent roadway), the roadway becomes a level tangent, and CMF becomes 1.0.

### HORIZONTAL CURVES AND TANGENTS AT TYPE 1 SAG VERTICAL CURVES

CMFs for horizontal curves and tangents at type 1 sag vertical curves can be derived from figure 25 and figure 26 as shown in figure 47 and figure 48\*:

$$CMF_{SI,FI} = \left\{ \begin{array}{ll} \exp \left[ 10.51 \frac{1}{K} + 0.011 \left( \frac{5,730}{R} \right) \frac{L_{VC}}{K} \right] & \text{for horizontal curves} \\ \exp \left[ 10.51 \frac{1}{K} \right] & \text{for tangents at type 1 sags} \\ 1.0 & \text{for level tangents (base condition)} \end{array} \right\}$$

**Figure 47. Equation. FI CMF for horizontal curves and tangents at type 1 sag vertical curves.**

$$CMF_{SI,PDO} = \left\{ \begin{array}{ll} \exp \left[ 8.62 \frac{1}{K} + 0.010 \left( \frac{5,730}{R} \right) \frac{L_{VC}}{K} \right] & \text{for horizontal curves} \\ \exp \left[ 8.62 \frac{1}{K} \right] & \text{for tangents at type 1 sags} \\ 1.0 & \text{for level tangents (base condition)} \end{array} \right\}$$

**Figure 48. Equation. PDO CMF for horizontal curves and tangents at type 1 sag vertical curves.**

The functional relationships shown in figure 25 (crashes/mi/year) and figure 47 (CMF) for FI crashes are illustrated in figure 49 for  $K$  values of 250, 125, 83, 63, and 50.  $R$  ranged from 100 to 11,460 ft, AADT was fixed at 2,000 vehicles/day, and  $L_{VC}$  was fixed at 500 ft. Similarly, figure 50 illustrates the relationships shown in figure 26 and figure 48 for PDO crashes.

To calculate CMF for FI or PDO crashes for a given horizontal curve at a type 1 sag vertical curve, the actual values of  $R$  (ft),  $L_{VC}$  (ft), and parameter  $K$  (ft/percent) are substituted in figure 47 or figure 48. Example CMFs were calculated for rural two-lane roadways with a vertical curve length of 500 ft, an  $R$  of 1,433 or 5,730 ft, and  $K$  values of 250, 125, 83, 63, and 50 using figure 47 and figure 48. The results are shown in table 19 for rural two-lane roadways with AADTs from 200 to 26,000 vehicles/day.

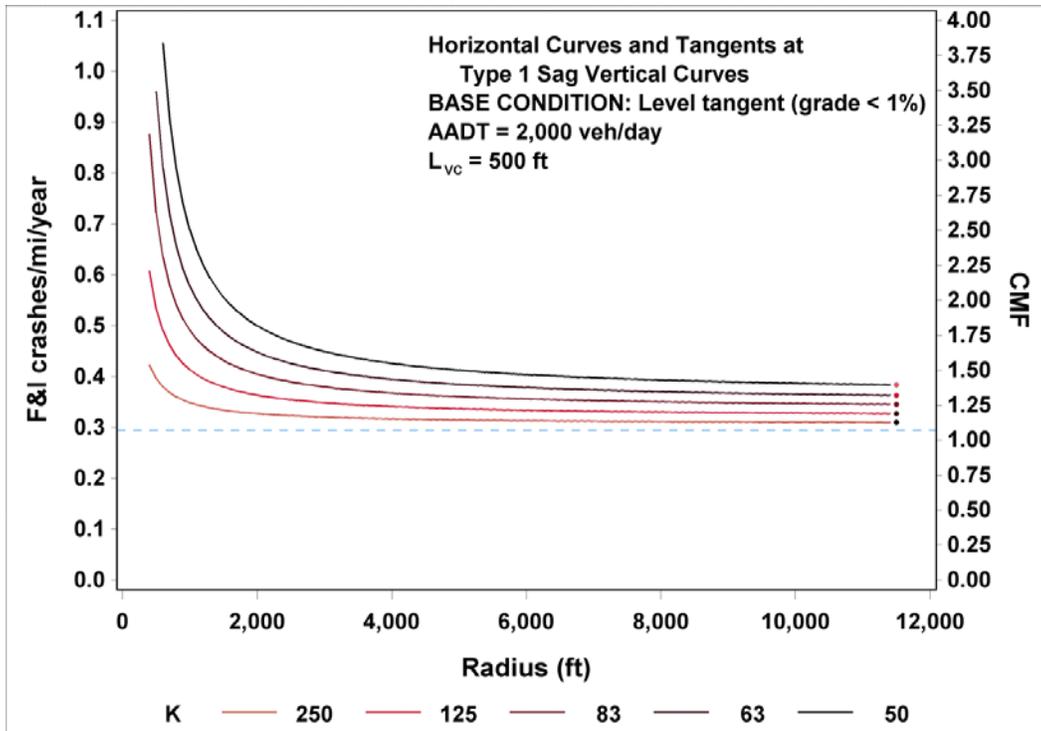


Figure 49. Graph. Predicted FI crashes/mi/year and CMFs for horizontal curves and tangents at type 1 sag vertical curves.

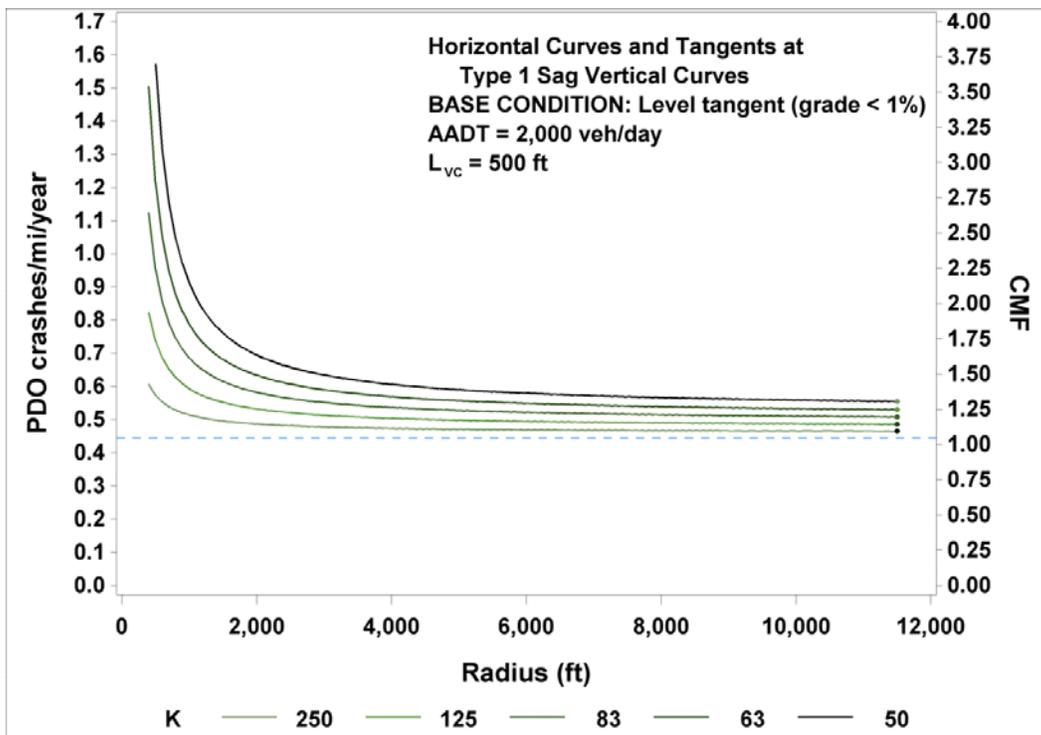


Figure 50. Graph. Predicted PDO crashes/mi/year and CMFs for horizontal curves and tangents at type 1 sag vertical curves.

**Table 19. Example CMFs for FI and PDO crashes on horizontal curves and tangents at type 1 sag vertical curves.**

K	FI CMFs			PDO CMFs		
	Tangent at Sag	R = 1,433 ft	R = 5,730 ft	Tangent at Sag	R = 1,433 ft	R = 5,730 ft
250	1.04	1.14	1.07	1.04	1.12	1.05
125	1.09	1.30	1.14	1.07	1.25	1.11
83	1.13	1.49	1.21	1.11	1.39	1.17
63	1.18	1.68	1.29	1.15	1.55	1.24
50	1.23	1.93	1.38	1.19	1.74	1.31

In any given column in table 19, CMFs show the increasing effect on crashes of decreasing *K* (sharper sag) for a horizontal curve or tangent at a type 1 sag vertical curve. This effect is slightly less pronounced for PDO than for FI crashes. When *K* becomes very large (i.e., level roadway), and *R* becomes infinite (i.e., tangent roadway), the roadway becomes a level tangent, and CMF becomes 1.0.

**HORIZONTAL CURVES AND TANGENTS AT TYPE 2 CREST VERTICAL CURVES**

CMFs for horizontal curves and tangents at type 2 crest vertical curves can be derived from figure 32 and figure 33 as shown in figure 51 and figure 52\*:

$$CMF_{C2,FI} = \begin{cases} \exp \left[ 0.20 \ln \left( 2 \times \frac{5,730}{R} \right) \right] & \text{for horizontal curves} \\ 1.0 & \text{for tangents at type 2 crests} \\ 1.0 & \text{for level tangents (base condition)} \end{cases}$$

**Figure 51. Equation. FI CMF for horizontal curves and tangents at type 2 crest vertical curves.**

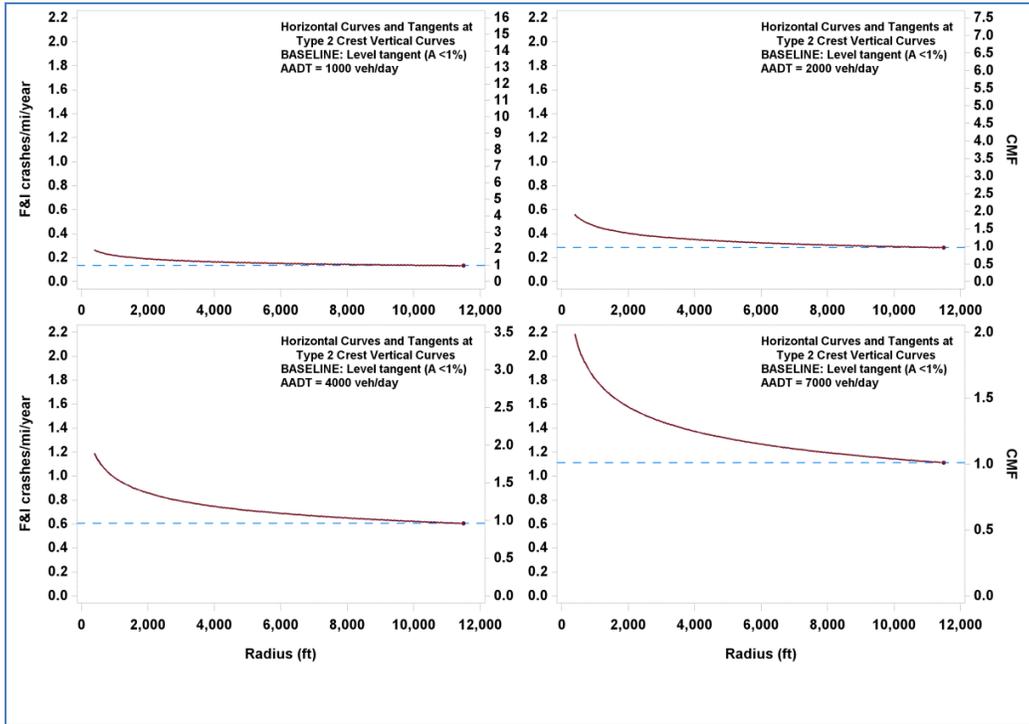
$$CMF_{C2,PDO} = \begin{cases} \exp \left[ 0.10 \ln \left( 2 \times \frac{5,730}{R} \right) \right] & \text{for horizontal curves} \\ 1.0 & \text{for tangents at type 2 crests} \\ 1.0 & \text{for level tangents (base condition)} \end{cases}$$

**Figure 52. Equation. PDO CMF for horizontal curves and tangents at type 2 crest vertical curves.**

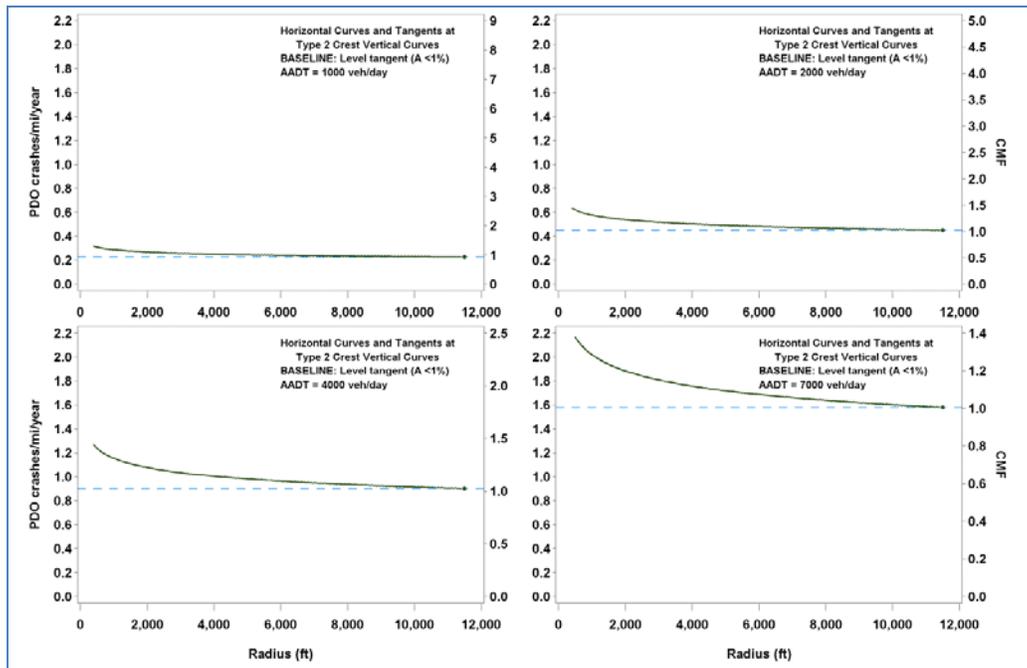
The functional relationships shown in figure 32 (crashes/mi/year) and figure 51 (CMF) for FI crashes are illustrated in figure 53 for an *R* range from 100 to 11,460 ft and AADTs of 1,000, 2,000, 4,000, and 7,000 vehicles/day (corresponding approximately to the 25th, 50th, 75th, and 90th percentile AADTs for rural two-lane highways in the database). Similarly, figure 54 illustrates the relationships shown in figure 33 and figure 52 for PDO crashes.

To calculate CMF for FI or PDO crashes for a given horizontal curve at a type 2 crest vertical curve, the actual value of *R* (ft) is substituted in figure 51 or figure 52. Example CMFs were calculated for rural two-lane roadways with an *R* of 1,433, 5,730, or 11,460 ft using figure 51

and figure 52. The results are shown in table 20 for rural two-lane roadways with AADTs from 200 to 26,000 vehicles/day.



**Figure 53. Graph. Predicted FI crashes/mi/year and CMFs for horizontal curves and tangents at type 2 crest vertical curves.**



**Figure 54. Graph. Predicted PDO crashes/mi/year and CMFs for horizontal curves and tangents at type 2 crest vertical curves.**

**Table 20. Example CMFs for FI and PDO crashes on horizontal curves and tangents at type 2 crest vertical curves.**

FI CMFs			PDO CMFs		
R = 1,433 ft	R = 5,730 ft	R = 11,460 ft	R = 1,433 ft	R = 5,730 ft	R = 11,460 ft
1.52	1.15	1.00	1.23	1.07	1.00

Only R of the horizontal curve at a type 2 vertical crest has an effect on crash rates, with increasing radii producing smaller CMFs. The effect of R is more pronounced for FI crashes than for PDO crashes. When R becomes infinite, the roadway becomes a tangent, and CMF becomes 1.0 regardless of the vertical curve characteristics.

Because this CMF lacks any measure of vertical alignment, consideration may be given to replacing this CMF with a CMF based on figure 10 and figure 11.

### HORIZONTAL CURVES AND TANGENTS AT TYPE 2 SAG VERTICAL CURVES

CMFs for horizontal curves and tangents at type 2 sag vertical curves can be derived from figure 37 and figure 38 as shown in figure 55 and figure 56\*:

$$CMF_{S2,FI} = \begin{cases} \exp \left[ 0.188 \ln \left( 2 \times \frac{5,730}{R} \right) \right] & \text{for horizontal curves} \\ 1.0 & \text{for tangents at type 2 sags} \\ 1.0 & \text{for level tangents (base condition)} \end{cases}$$

**Figure 55. Equation. FI CMF for horizontal curves and tangents at type 2 sag vertical curves.**

$$CMF_{S2,PDO} = \begin{cases} \exp \left[ 0.022 \left( \frac{5,730}{R} \right) A \right] & \text{for horizontal curves} \\ 1.0 & \text{for tangents at type 2 sags} \\ 1.0 & \text{for level tangents (base condition)} \end{cases}$$

**Figure 56. Equation. PDO CMF for horizontal curves and tangents at type 2 sag vertical curves.**

The functional relationships shown in figure 37 (crashes/mi/year) and figure 55 (CMF) for FI crashes are illustrated in figure 57 for an R range from 100 to 11,460 ft and AADTs of 1,000, 2,000, 4,000, and 7,000 vehicles/day. Similarly, figure 58 illustrates the relationships shown in figure 38 and figure 56 for PDO crashes for K values of 250, 125, 83, 63, and 50; curve radii ranging from 100 to 11,460 ft; and AADT fixed at 2,000 vehicles/day.

To calculate CMF for FI crashes for a given horizontal curve at a type 2 sag vertical curve, the actual value of R (ft) is substituted in figure 55. To calculate CMF for PDO crashes, the actual value of R (ft),  $L_{VC}$ , and K are substituted in figure 56. Example FI CMFs were calculated for rural two-lane roadways with an R of 1,433, 5,730, or 11,460 ft using figure 55. Example PDO CMFs were calculated using figure 56 for rural two-lane roadways with an R of 1,433 or 5,730 ft, an  $L_{VC}$  of 500 ft, and A values ranging from 2 to 10 percent. The results are shown in

table 21 for FI crashes and table 22 for PDO crashes for rural two-lane roadways with AADTs from 200 to 26,000 vehicles/day.

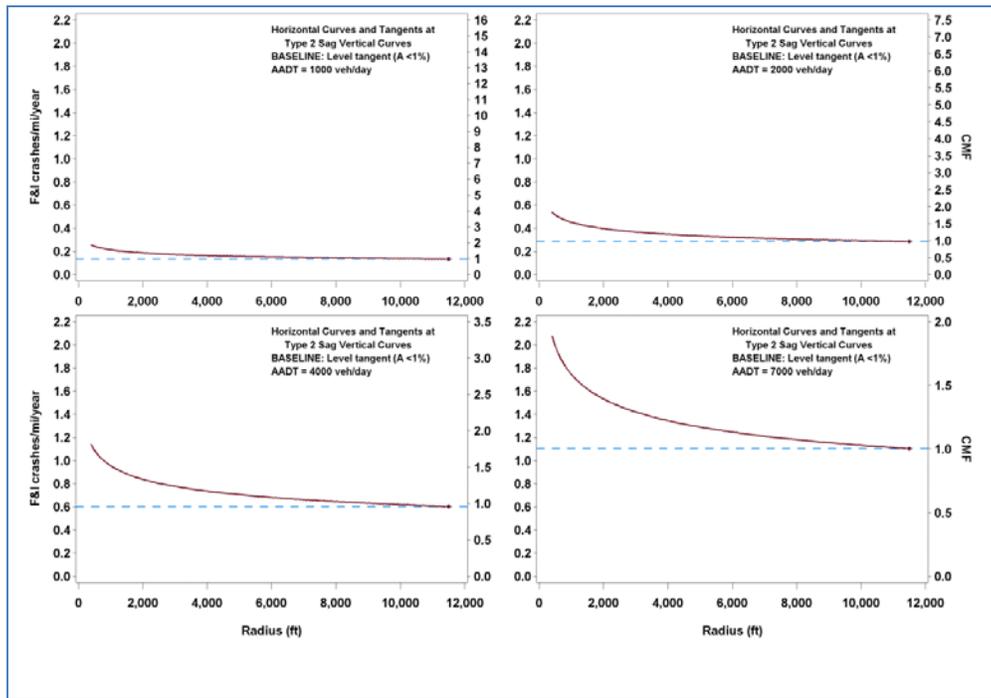


Figure 57. Graph. Predicted FI crashes/mi/year and CMFs for horizontal curves and tangents at type 2 sag vertical curves.

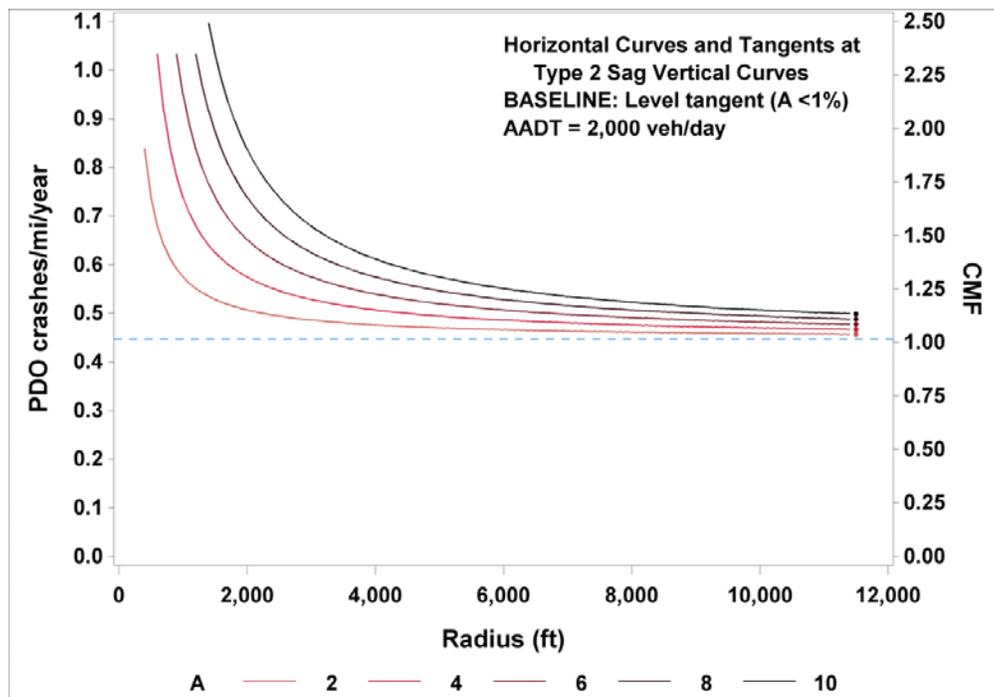


Figure 58. Graph. Predicted PDO crashes/mi/year and CMFs for horizontal curves and tangents at type 2 sag vertical curves.

**Table 21. Example CMFs for FI crashes on horizontal curves and tangents at type 2 sag vertical curves.**

FI CMFs		
R = 1,433 ft	R = 5,730 ft	R = 11,460 ft
1.48	1.14	1.00

**Table 22. Example CMFs PDO crashes on horizontal curves and tangents at type 2 sag vertical curves.**

A	PDO CMFs		
	R = 1,433 ft	R = 5,730 ft	R = 11,460 ft
All tangents	1.00	1.00	1.00
2	1.19	1.04	1.02
4	1.42	1.09	1.04
6	1.69	1.14	1.07
8	2.02	1.19	1.09
10	2.40	1.24	1.12

Only R of the horizontal curve at a type 2 vertical sag has an effect on FI crash rates, with increasing radii producing smaller CMFs (see table 21). When R becomes infinite, the roadway becomes a tangent and CMF becomes 1.0 regardless of the characteristics of the vertical curve. Only the interaction term of A and inverse R has an effect on PDO crash rates (see table 22), highlighting the joint effect of sharp horizontal curves at steep vertical curves on PDO crashes. When the initial and final grade difference, A, nears zero (i.e., level roadway) and the R becomes infinite (i.e., tangent roadway), the roadway becomes a level tangent, and CMF becomes 1.0.

Because this CMF (at least for FI crashes) lacks any measure of vertical alignment, consideration may be given to replacing this CMF with a CMF based on figure 10 and figure 11.

### CMFs FOR COMBINED CRASH SEVERITY LEVELS

This study provides separate CMFs for FI and PDO crashes. At some future time, the AASHTO HSM may be structured to provide separate CMFs by severity level for all CMFs. In the meantime, if users want a CMF for total crashes (i.e., all crash severity levels combined), it can be computed from the results in this report as shown in figure 59\*:

$$CMF_{TOT} = [(CMF_{FI} - 1.0) \times P_{FI} + (CMF_{PDO} - 1.0) \times P_{PDO}] + 1.0$$

**Figure 59. Equation. CMF for combined crash severity level.**

Where:

$CMF_{TOT}$  = CMF for total crashes (i.e., all severity levels combined).

$CMF_{FI}$  = CMF for FI crashes.

$CMF_{PDO}$  = CMF for PDO crashes.

$P_{FI}$  = FI crashes expressed as a proportion of total crashes.

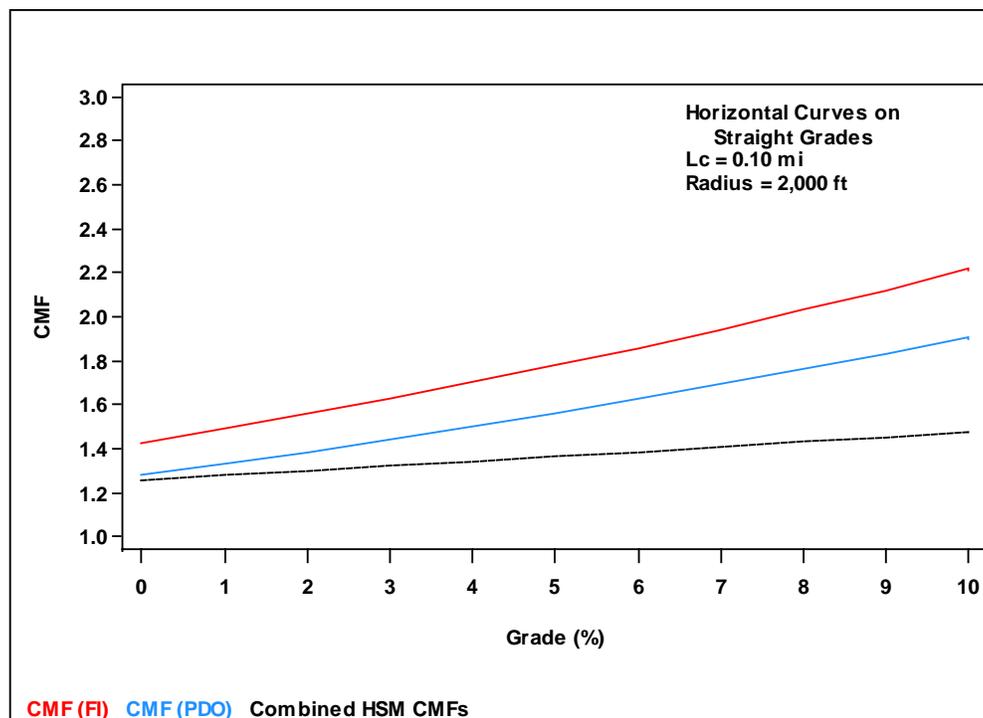
$P_{PDO}$  = PDO crashes expressed as a proportion of total crashes.

Values used for  $P_{FI}$  and  $P_{PDO}$  must always sum to 1.0. Values of  $P_{FI}$  and  $P_{PDO}$  indicated for rural two-lane highways in AASHTO HSM table 10-3 ( $P_{FI} = 0.321$  and  $P_{PDO} = 0.679$ ) may be used, or users may develop values for  $P_{FI}$  and  $P_{PDO}$  from their agencies' data.<sup>(1)</sup>

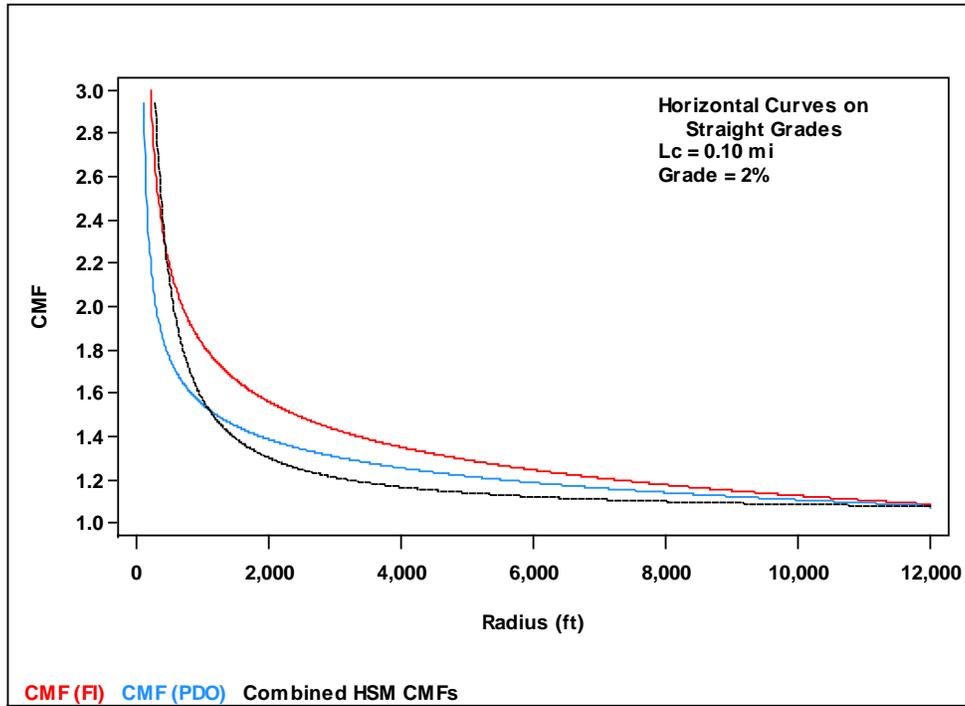
### COMPARISON OF RESULTS TO EXISTING HSM CMFS

The current AASHTO HSM presents separate CMFs for horizontal curves and straight grades, as seen as  $CMF_{HC}$  in figure 3 and  $CMF_G$  in figure 5, respectively.<sup>(1)</sup> The combined effect of horizontal curves and grades is represented in the AASHTO HSM as the product of  $CMF_{HC}$  and  $CMF_G$ . Figure 60 illustrates a comparison of CMFs for horizontal curves on straight grades developed in this study, and shown in figure 39 and figure 40 for FI and PDO crashes, respectively, to the combined AASHTO HSM CMF, holding length of horizontal curve and radius constant while varying percent grade. Figure 61 is an analogous plot, keeping the length of horizontal curve and percent grade constant while varying the radius of horizontal curve.

The plots show that the new CMF for FI crashes is consistently larger than the new CMF for PDO crashes. This represents an advance in knowledge over the AASHTO HSM, which treats CMFs for all severity levels as equal.<sup>(1)</sup> The plots also show that the new CMFs are generally larger than the combined AASHTO HSM CMFs, except that the new CMF for PDO crashes is smaller than the existing CMFs for horizontal curves with short radii. No other comparisons between the HSM CMFs and the CMFs developed in this study are relevant because the AASHTO HSM does not address the safety effects of crest or sag vertical curves.



**Figure 60. Graph. Comparison of CMFs developed in this study to the combined AASHTO HSM CMFs for horizontal curves and grades for fixed radius and varying percent grades.<sup>(1)</sup>**



**Figure 61. Graph. Comparison of CMFs developed in this study to the combined AASHTO HSM CMFs for horizontal curves and grades for fixed percent grade and varying radii.<sup>(1)</sup>**

## CHAPTER 6—CONCLUSIONS AND RECOMMENDATIONS

This chapter presents the conclusions and recommendations based on the results of the study.

### CONCLUSIONS

Conclusions for this study are as follows:

- For tangents and horizontal curves on straight grades, prediction models for crash frequency by severity level are presented in figure 10 and figure 11, with parameter estimates presented in table 8. These models include a main effect for AADT, a main effect for horizontal curve radius, a main effect for percent grade, and an interaction between horizontal curve radius and length of curve. The models indicate that crash frequency increases with decreasing horizontal curve radius, decreasing horizontal curve length, and increasing percent grade. The interaction term shows that short sharp horizontal curves are associated with higher crash frequencies. CMFs corresponding to the crash prediction models are presented in figure 39, figure 40, and table 17.
- For tangents and horizontal curves at type 1 crest vertical curves, prediction models for crash frequency by severity level are presented in figure 16 and figure 17, with parameter estimates presented in table 10. Alternate prediction models are shown in figure 20 and figure 21. These models include a main effect for AADT and an interaction between horizontal curve radius and the difference between initial and final grade (alternatively, an interaction between horizontal curve radius, vertical curve length, and  $K$ ). The models indicate that crash frequency increases with decreasing horizontal curve radius and increases with increasing grade difference. The interaction term shows that short horizontal curves at sharp crest vertical curves are associated with higher crash frequencies. CMFs corresponding to the crash prediction models are presented in figure 43, figure 44, and table 18.
- For tangents and horizontal curves at type 1 sag vertical curves, prediction models for crash frequency by severity level are presented in figure 23 and figure 24, with parameter estimates presented in table 12. Alternate prediction models are shown in figure 27 and figure 28. These models include a main effect for AADT, a main effect for  $K$ , and an interaction between horizontal curve radius and the difference between initial and final grade (alternatively, an interaction between horizontal curve radius, vertical curve length, and  $K$ ). The models indicate that crash frequency increases with decreasing  $K$ , decreasing horizontal curve radius, and increasing grade difference. The interaction term shows that short horizontal curves at sharp sag vertical curves are associated with higher crash frequencies. CMFs corresponding to the crash prediction models are presented in figure 47, figure 48, and table 19.
- For tangents and horizontal curves at type 2 crest vertical curves, prediction models for crash frequency by severity level are presented in figure 30 and figure 31, with parameter estimates presented in table 14. These models include only two main effects: a main effect for AADT and a main effect for horizontal curve radius. The models indicate that

crash frequency increases with decreasing horizontal curve radius. CMFs corresponding to the crash prediction models are presented in figure 51, figure 52, and table 20. Consideration will be given to using figure 10 and figure 11, with grade set equal to the average of  $G_1$  and  $G_2$  in place of figure 30 and figure 31.

- For tangents and horizontal curves at type 2 sag vertical curves, prediction models for crash frequency by severity level are presented in figure 35 and figure 36, with parameter estimates presented in table 16. The FI crash prediction model includes only two main effects: a main effect for AADT and a main effect for horizontal curve radius. This model indicates that FI crash frequency increases with decreasing horizontal curve radius. The PDO crash prediction model includes a main effect for AADT, an interaction between horizontal curve radius, and the difference between  $G_1$  and  $G_2$ . The PDO model indicates that crash frequency increases with decreasing horizontal curve radius and increases with increasing grade difference. The interaction term shows that short horizontal curves at sharp sag vertical curves are associated with higher crash frequencies. CMFs corresponding to the crash prediction models are presented in figure 55, figure 56, table 21, and table 22. Consideration will be given to using figure 10 and figure 11 with grade set equal to the average of  $G_1$  and  $G_2$  in place of figure 30 and figure 17.
- For all five horizontal and vertical alignment combinations, the coefficient of the AADT term in the crash prediction models is nearly equal to 1.0, indicating that crash frequencies are proportional to AADT.
- The AADT coefficients in the models range from 0.99 to 1.10. Consideration will be given as to whether the AADT coefficient should be set equal to 1.0 for consistency with the current AASHTO HSM.<sup>(1)</sup>

## RECOMMENDATIONS

Recommendations based on the study results are as follows:

- CMFs developed in this report should be considered for incorporation in the next edition of the AASHTO HSM.
- It would be desirable to validate the CMFs developed in this report using data from one or more additional States. The current emphasis being placed on roadway inventories for asset management may lead to additional suitable databases.
- Ultimately, it would be desirable to have safety prediction models for rural two-lane highways for use in the AASHTO HSM that consider the safety effects of a full range of variables of interest and all of their combinations and interactions. This will require more comprehensive databases than those currently available.

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