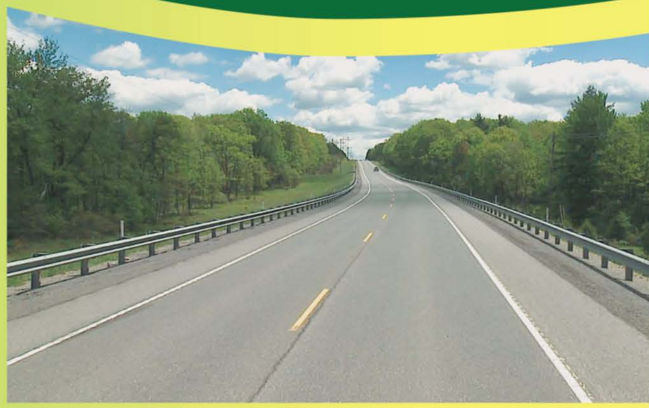
Reducing Roadway Departure Crashes at Horizontal Curve Sections on Two-Lane Rural Highways



FHWA Safety Program

U.S. Department of Transportation
Federal Highway Administration

Safe Roads for a Safer Future
Investment in roadway safety saves lives

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REDUCING ROADWAY DEPARTURE CRASHES AT HORIZONTAL CURVE SECTIONS ON TWO-LANE RURAL HIGHWAYS

FINAL REPORT

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16. Abstract

While several proven safety countermeasures and strategies exist to mitigate roadway departure crashes on horizontal curve sections of two-lane rural highways, there are several countermeasures or strategies that have yet to be proven via a rigorous safety evaluation. The purpose of this research was to identify several of these strategies or countermeasures, and to perform a statistical assessment of their safety effectiveness. This report details the following three evaluations: (1) observational before-after study of curve ahead warning pavement markings, (2) cross-sectional study of delineators on guiderail along horizontal curves, and (3) cross-sectional study of the safety effects of geometric design consistency. The findings from these evaluations indicate that the expected number of roadway departure crashes are associated with the horizontal curve radius, radii of adjacent horizontal curves, and the tangent lengths between curves. Further, the expected number of roadway departure crashes is associated with side friction demand on horizontal curves. Guiderail with delineators are expected to reduce total, fatal plus injury, run-off-road (ROR), and nighttime crashes along horizontal curves that are four degrees or sharper. Horizontal curve warning pavement markings are associated with fewer expected total, fatal plus injury, ROR, nighttime, nighttime ROR, and nighttime fatal plus injury crashes on two-lane rural highways.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

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

APPROXIMATE CONVERSIONS FROM SI UNITS

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

*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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LIST OF ABBREVIATIONS

AADT	average annual daily traffic
AASHTO	American Association of State Highway and Traffic Officials
ANOVA	univariate analyses of variance
ASMD	absolute standardized mean difference
ATE	average treatment effect
ATT	average treatment effect for the treated
ATU	average treatment effect for the untreated
CMF	crash modification factor
CRR-M	modified change radius rate
CTRE	Center for Transportation Research and Education
DSDS	dynamic speed display signs
DSFS	dynamic speed feedback signs
EB	empirical Bayes
FB	full Bayes method
FHWA	Federal Highway Administration
FI	fatal and injury crashes
GD	guiderail with delineators
GIS	geographic information systems
G only	guiderail only
HFST	high friction surface treatment
HPMS	Highway Performance Monitoring System
HSIS	Highway Safety Information System
HSM	Highway Safety Manual
IHSDM	Interactive Highway Safety Design Model
IRI	International Roughness Index
K-S	Kolmogorov-Smirnov test
LED	light-emitting diode
MLE	maximum likelihood estimation
MUTCD	Manual on Uniform Traffic Control Devices
NB	negative binomial

NDS	Naturalistic Driving Study
NN	nearest neighbor matching
PC	point of curvature
PDO	property damage only
PennDOT	Pennsylvania Department of Transportation
PMD	post-mounted delineator
PT	point of tangency
QQ	quantile-quantile
RwD-A	incapacitating injury roadway departure crashes
RwD-B	non-incapacitating injury
RwD-C	possible injury
RwD-K	fatal roadway departure crashes
RwD-KABC	fatal-plus-injury roadway departure crashes
RwD-KABCO	total (i.e., all severities) roadway departure crashes
RwD-O	no injury
RID	Roadway Information Database
ROR	run-off-the-road
RTR	ratio of tangent length to radius
SB	standardized bias
SDCWS	sequential dynamic curve warning system
SHRP	Strategic Highway Research Program
SPF	safety performance function
SVROR	single-vehicle run-off-road
TDR	traffic data recorder
UDOT	Utah Department of Transportation
VIEDA	Variable Introduction Exploratory Data Analysis

CHAPTER 1. INTRODUCTION

Roadway departure crashes are defined as those in which a vehicle "crosses an edge line or a centerline, or otherwise leaves the traveled way." (FHWA 2014) In 2016, there were approximately 7.28 million police-reported crashes, resulting in 37,461 fatalities and more than 3.1 million injuries, on highways and streets in the United States. (NHTSA 2018) The Federal Highway Administration (FHWA) estimates that more than 50 percent of traffic fatalities result from roadway departure crashes. Vehicles are more likely to depart the roadway at locations where the alignment changes. Research by Glennon et al. (1985) estimated that the average crash rate along horizontal curve sections of two-lane rural highways is three times higher than on tangent roadway sections, and that the severity of roadway departure crashes along horizontal curves is greater than the severity of roadway departure crashes along tangent roadway sections.

A Guide for Reducing Collisions on Horizontal Curves further illustrates the problem associated with roadway departure crashes along horizontal curves. (Torbic et al. 2004) The authors' found that nearly 25 percent of people who die each year on the Nation's roadways are killed in vehicle crashes at curves. About 75 percent of all fatal crashes occur in rural areas, and approximately 76 percent of the curve-related fatal crashes involve single-vehicles leaving the roadway and striking trees, utility poles, rocks, or other fixed objects or overturning. FHWA (2014) estimates that more than 70 percent of fatal roadway departure crashes involve overturning vehicles, opposite direction crashes, or collisions with trees. Among the risk factors for these three crash types are the following:

- **Overturn Crashes:** 76 percent are in rural areas, 72 percent occur on roads with posted speed limits of 50 mph or higher, and 43 percent are reported on horizontal curve sections.
- **Opposite Direction Crashes:** 83 percent are on undivided roads, 68 percent occur in rural areas, 68 percent occur on roadways with posted limits of 50 mph or higher, and 32 percent are reported on horizontal curve sections.
- **Roadside Trees and Shrubs:** 68 percent are in rural areas, 52 percent occur on roads with posted speed limits of 50 mph or higher, and 46 percent are reported on horizontal curve sections.

Collectively, these statistics indicate that roadway departure crashes along horizontal curves on two-lane rural highways are a significant safety issue. To mitigate this crash type, safety countermeasures aim to keep vehicles on the roadway, reduce the likelihood of a crash when vehicles leave the roadway or cross into opposing travel lanes, or reduce the severity of a roadway departure crash.

STRATEGIC APPROACH TO ROADWAY DEPARTURE CRASHES

In 2013, the FHWA developed a roadway departure strategic plan (FHWA 2017a). The areas of emphasis for the plan are overturning crashes, opposite direction crashes, and collisions with roadside trees and shrubs. Table 1 provides a list of strategies that are recommended to support the safety emphasis areas.

Table 1. Roadway departure strategies to support crash emphasis areas.

Emphasis Area	Strategies to Keep Vehicles on Road	Strategies to Reduce Potential for Crashes	Strategies to Minimize Crash Severity
Overturn Crashes	• Curve delineation • Friction treatments in curves • Edge line or shoulder rumble strips	• Safety Edge$_{SM}$ • Maintain clear zones • Traversable roadside slopes	• Barriers to shield fixed objects and slopes
Opposite Direction Crashes	• Centerline rumble strips • Friction treatments in curves	• Increase separation between opposing lanes in curves	• Median barriers
Roadside Trees and Shrubs	• Edge line and shoulder rumble strips • Curve delineation • Friction treatments in curves	• Clear zone improvements and maintenance, particularly along the outside of horizontal curves	• Barriers to shield fixed objectives

In order to address horizontal curve locations with roadway departure crash issues, the FHWA Office of Safety has identified several proven countermeasures. A number of these countermeasures are infrastructure-based treatments or strategies that are intended to reduce serious injuries and fatalities on horizontal curves of two-lane rural highways. The treatments or strategies include the following:

- **Roadside Design Improvements Along Horizontal Curves.** This strategy includes several treatments that are designed to either provide for a safe recovery in the event of a roadway departure, or to reduce the impact severity in a roadway departure crash. The treatments include: (1) an unobstructed, traversable clear zone, (2) slope flattening, (3) adding or widening shoulders, (4) cable barrier, (5) guardrail, and (6) concrete barrier.
- **Enhanced Delineation and Friction for Horizontal Curves**. This strategy includes several treatments to keep vehicles on the roadway, including enhanced delineation, such as pavement markings, post-mounted delineators, larger and more visible signs, and dynamic or flashing sequential signs. Additionally, restoring pavement friction and applying the high-friction surface treatment (HFST) is identified as a roadway surface roadway departure safety treatment.

- **Longitudinal Rumble Strips and Stripes**. Installation of rumble strips along the edge line, shoulder, or centerline is a strategy to keep the vehicle on the roadway.
- **Safety Edge$_{SM}$.** This treatment produces a pavement edge of approximately 30 degrees relative to the pavement cross-slope during the paving process, enabling vehicles that depart the roadway to more effectively recover.

Among the safety countermeasures or strategies identified above, several have crash reduction benefits that are documented based on empirical research. These benefits are shown in table 2.

Table 2. Crash reduction estimates for proven roadway departure countermeasure treatments or strategies (FHWA 2018).

Treatment of Strategy	Crash Type	Crash Reduction Estimate (percent)	Source
Increase clear zone from 3.3 to 16.7 ft	Total crashes	22	FHWA (2017b)
Increase clear zone from 16.7 to 30 ft	Total crashes	44	FHWA (2017b)
Install chevron signs	Fatal and injury crashes	16	FHWA (2017c)
Install chevron signs	Lane departure crashes at night	25	FHWA (2017c)
HFST	Total crashes	24	FHWA (2017c)
HFST	Wet road crashes	52	FHWA (2017c)
Shoulder rumble strips	Single-vehicle run-off road crashes	15	FHWA (2017d)
Shoulder rumble strips	Single-vehicle run-off-road fatal and injury crashes	29	FHWA (2017d)
Centerline rumble strips	Total crashes	9	FHWA (2017d)
Centerline rumble strips	Fatal and injury crashes	12	FHWA (2017d)
Centerline rumble strips	Total target (head-on and opposite direction sideswipe) crashes	30	FHWA (2017d)
Centerline rumble strips	Fatal and injury target crashes	44	FHWA (2017d)
Safety Edge$_{SM}$	Fatal and injury crashes	11	FHWA (2017e)

OBJECTIVES OF CURRENT STUDY

While several proven safety countermeasures and strategies exist to mitigate roadway departure crashes on horizontal curve sections of two-lane rural highways, there are several countermeasures or strategies that have yet to be proven via a rigorous safety evaluation. The purpose of this research was to identify several of these strategies or countermeasures, and to perform a statistical assessment of their safety effectiveness. This report details the following three evaluations:

- Observational before-after study of curve ahead warning pavement markings.
- Cross-sectional study of delineators on guiderail along horizontal curves.
- Cross-sectional study of the safety effects of geometric design consistency.

ORGANIZATION OF THE REPORT

The remainder of the report is organized into six subsequent chapters:

- Chapter 2 is a synthesis of literature related to the safety and operational effects of treatments intended to mitigate roadway departure crashes along horizontal curves of two-lane rural highways.
- Chapter 3 provides an overview of the three safety evaluations that were completed in the present study.
- Chapters 4 and 5 summarize two cross-sectional studies that evaluated the relationship between geometric design consistency and safety on two-lane rural highways.
- Chapter 6 describes a propensity scores-potential outcomes evaluation of delineators on guiderail along horizontal curves.
- Chapter 7 describes the methodology and results of a horizontal curve ahead warning pavement markings evaluation.
- Chapter 8 contains the overall findings from the research effort, as well as recommendations for future research.

CHAPTER 2. LITERATURE REVIEW

This chapter of the report is organized into three sections. The first describes two publications that were published as part of the current study, both of which include countermeasures that are intended to mitigate roadway departure crashes along horizontal curves of two-lane rural highways. The second section summarizes several Federal Highway Administration (FHWA) publications that were developed as part of other research efforts, but also provides information that supports the roadway departure program. The third section is a summary of existing research describing the relationship between traffic control devices and other roadway features, and their association with driver speed choice and safety.

LOW-COST TREATMENTS FOR HORIZONTAL CURVE SAFETY

In addition to the evaluations described in Chapters 4, 5, and 6 of this report, two other publications were developed by the research team to support the FHWA roadway departure program. FHWA provided the overall framework for the *Low-Cost Treatments for Horizontal Curve Safety 2016* report, which presents summary information for countermeasures that transportation agencies may consider applying on two-lane highways experiencing roadway departure crashes along horizontal curves. (Albin et al. 2016) For each countermeasure, the following information is documented (if available):

- General description of the countermeasure.
- Design elements or materials to use when deploying the countermeasure.
- How to apply the countermeasure.
- The effectiveness of the countermeasure.
- The relative cost (low, medium, or high) of the countermeasure.

The guide offers two strategies for implementing safety countermeasures, including the site-specific approach and the systemic approach. The countermeasures are organized into several basic categories, including: pavement markings, signs, pavement surface countermeasures, roadside improvements, and treatments along curves within at-grade intersections. In FHWA's *Low-Cost Treatments for Horizontal Curve Safety 2016* there are eight case study examples of safety improvement programs and countermeasures included in the guide. (Albin et al. 2016)

One sign application with potential safety improvements along horizontal curves on two-lane rural highways was the Sequential Dynamic Curve Warning System (SDCWS). The SDCWS is a series of horizontal curve chevron signs with solar-powered flashing lights embedded in the signs. The flashing lights can be simultaneous or a sequence of lights moving toward or away from the driver. A driving simulator study at the University of Utah's Utah Traffic Lab identified several SDCWS settings that impacted operating speeds and lane keeping behavior along a simulated version of a two-lane rural highway in Pennsylvania with 17 horizontal curves. Sixty-eight participants were recruited to participate in the study to examine effective flash rates, speed activation thresholds, and flashing sequences when deploying SDCWS along horizontal curves on two-lane rural highways. Based on the outcome of the simulator study, an outdoor field study was completed at three sites in Wisconsin to further assess the potentially effective flash rates and flashing patterns identified in the simulator study. Four different conditions were studied in the field, including a speed-activation threshold that was either 5 or 10 mph above the curve advisory speed, two different flash rates (simultaneous and away from the driver), and two different flashing patterns (three flashes per second and one flash per second).

Based on the indoor driving simulator study, a flashing sequence away from the driver at a low flash rate (1 Hz), or a simultaneous flashing pattern with a high flash rate produced the greatest speed reduction effects on drivers approaching horizontal curves at higher speeds on two-lane rural highways. The flashing sequence away from the driver also produced the lowest probability of a lane departure event. These two conditions were then evaluated in the field at three locations in Wisconsin. In addition to the flashing pattern and flash rate, the speed-activation threshold was also varied in the field to either 5mph or 10mph above the curve advisory speeds.

The field study found that, among the conditions tested, the simultaneous flashing pattern, set to activate at a rate of 3 Hz when a driver approached the curve more than 5 mph above the curve advisory speed, produced the most desirable outcome based on the speed metrics considered in the present study. However, a flashing pattern away from the driver with a flashing rate of 1 Hz set to activate when approach speeds exceed the curve advisory speed by any amount appears to be another effective setting when the present field study results are assessed in conjunction with findings from the driving simulator study and a previous study by Smadi et al. (2015)

OTHER FHWA ROADWAY DEPARTURE SAFETY MANAGEMENT RESOURCES

In addition to the two reports identified above, FHWA published four other reports that support the roadway departure program. Each of these is summarized below.

Factors Influencing Speed and Safety on Rural and Suburban Roads

This informational report summarizes a variety of treatments that to support speed management along horizontal curves and tangents of rural and suburban roads. (Boodlal et al. 2015) Included is a detailed review of design features and current practices associated with methods to manage operating speeds. The remainder of the report describes three studies undertaken to evaluate the: (1) operational effects of the high-friction surface treatment on horizontal curves of two-lane rural highways, (2) operational effects of optical speed bars on rural and suburban roads, and (3) safety effects of lane and shoulder width combinations on two-lane rural highways.

The main findings from this research effort are as follows:

- No consistent differences in various driver speed metrics could be found after a high-friction surface was applied to horizontal curves on two-lane rural highways. Friction tests confirmed that the friction supply increased considerably at the locations treated with the high-friction surface treatment, and that the friction levels remained high for a period one year after the treatment was applied – no additional friction measurements were recorded.
- A before-after field study of the optical speed bar treatment showed that this treatment was not associated with statistically significant changes in driver operating speeds.

- The results of the lane and shoulder with safety evaluation showed complex interactions between expected crash frequency and cross-section dimensions for rural, two-lane roads. For narrower total paved widths, the safety evaluation found that the optimal lane width appears to be 12 ft. As total paved widths become larger, there is not necessarily a safety benefit from using a wider lane, and in some cases, using a narrower lane appears to result in lower expected crash frequencies.

Integrating Speed Management within Roadway Departure, Intersection, and Pedestrian and Bicyclist Safety

Neuner et al. (2016) identifies current practices related to speed management within the FHWA key focus areas of roadway departure, intersection, and pedestrian safety. In the roadway departure focus area, national trends identified overturn or rollover crashes, opposite direction crashes, and collisions involving trees as the most harmful event in 75 percent of roadway departure crashes. Other trend analyses found that rural areas, roadways with posted speed limits equal to 50 mph or greater, horizontal curves, and adverse weather conditions are locations in which roadway departure crashes frequently occur. Several strategies were identified to mitigate roadway departure crashes, including the following:

- Establishing appropriate speed limits.
- Deploy pavement marking, rumble strip, static or dynamic horizontal curve signing, high-friction surface treatment, Safety EdgeSM, or pavement widening safety countermeasures.
- Deploy safety countermeasures to reduce crash severity, such as removing, shielding, or delineating fixed objects; increasing the clear zone width, or flattening side slopes.

State of the Practice for Shoulder and Center Line Rumble Strip Implementation on Non-Freeway Facilities

Himes et al. (2017) summarized the current state-of-the-practice related to centerline and shoulder rumble strip use and developed a decision support guide to inform transportation agencies on the application of centerline and shoulder rumble strip installation. The literature review details research related to rumble strip design, noise and vibration characteristics associated with the designs, impacts on bicyclists and motorcyclists, pavement condition impacts, pavement marking (rumble stripe) visibility, and the operational and safety effectiveness of centerline and shoulder rumble strips. The report also describes agency practices regarding rumble strip installation criteria and patterns.

Development of Crash Modification Factors for the Application of the SafetyEdgeSM Treatment on Two-Lane Rural Roads

The SafetyEdgeSM is a low-cost treatment that is constructed using a paver attachment, enabling the pavement edge to be paved and compacted at a 30-degree angle. When this treatment is implemented, it permits a vehicle that has departed the paved roadway to make a smooth return to the travel lane after one or more wheels have left the pavement. An observational before-after safety evaluation developed crash modification factors for this treatment. A multi-State study found that drop-off related crashes are expected to decrease by approximately 34.5 percent; run-off-road related crashes are expected to decrease by approximately 21 percent; opposite direction crashes are expected to decrease by approximately 18.7 percent; and fatal plus injury crashes are expected to decrease by approximately 10.8 percent after implementing the SafetyEdgeSM on two-lane rural highways. (FHWA n.d.) A benefit-cost analysis found that the treatment can be applied cost-effectively.

Other Research Related to Roadway Departure Crashes

In addition to the research reports that are noted above, all of which support the FHWA roadway departure program, there is a large body of published research that also identifies methods or practices to manage vehicle operating speeds or safety along horizontal curves sections of two-lane rural highways. This literature is summarized below, and organized into sections that are related to treatments that are associated with operating speeds on horizontal curves, treatments that are related to roadway departure crashes on horizontal curves, and the relationship between roadway and roadside design features on operating speeds and safety.

Effectiveness of Traffic Signs, Pavement Markings, and Other Roadway Treatments in Reducing Operating Speeds on Horizontal Curves

Katz (2004) investigated the potential for peripheral transverse (i.e., optical speed bars) pavement markings to reduce vehicle operating speeds approaching and within horizontal curves. Each curve treatment had a slightly different pattern, which is illustrated in figure 1. The pattern spacing was based on a desired speed reduction from the approach tangent to the advisory speed for the horizontal curve. The speed bars were applied such that they intruded into the travel lane, perpendicular to the edge of the travel lane. Three curves were selected for inclusion in the study based on complaints that speeding had been cited as a safety problem. One curve was located on an interstate (in New York) and two were on two-lane rural highways (in Mississippi and Texas). No comparison site locations were included in the study. Vehicle operating speed data were collected before the peripheral transverse lines were installed, immediately after installation, and six months after installation. Speeds were collected at a location upstream of the curve and just prior to the point of curvature (PC) of the curve to permit speed reductions (i.e., rates of deceleration) to be computed.

The results indicated that deployment of the optical speed bar markings resulted in a statistically significant decrease in vehicle speeds at the curve speed measurement locations for both the New York and Mississippi test locations in the immediate and extended after periods. No statistically significant change in speed was found at the upstream data collection location for the New York or Mississippi horizontal curves for either after period. The Texas curve had a statistically significant decrease in speed at the upstream location but no statistical difference in either after period from the before period at the curve. Speed changes at the curve locations in the after period (adjusted using the differences in the upstream speeds) included a 9.5-mph decrease for the immediate after period and 4-mph decrease in the extended period for the New York curve, 4.6-mph decrease for the immediate after period and 1.8-mph decrease in the extended after period for the Mississippi curve, and a 13.6-mph increase for the immediate after period and 6.6-mph increase in the extended after period for the Texas curve.

Figure 1. Photo. Optical speed bars (Katz 2004).

The impacts of optical speed bars on vehicle operating speeds along horizontal curves were also investigated by Gates et al. (2008) The speed bars were 18 inches long by 12 inches wide and were painted perpendicular to the edge line of a single horizontal curve on Interstate 43 in Milwaukee, Wisconsin. Speeds were measured two months before, one week after, and six months after the speed bars were painted. Speeds were collected 850 ft upstream of the PC (350 ft upstream of the start of speed bars), 100 ft downstream of the PC, and 700 ft downstream of the PC (200 ft downstream from the end of the speed bars). No comparison locations were included in the observational before-after study.

The findings from the field study indicated that mean speed reductions of 0.05 to 5 mph were statistically significant when comparing the before to the immediate and long-term after periods. The findings also indicated that light condition may influence vehicle operating speeds, but the authors indicated that the magnitude was too small to be practically significant.

Speed bars that were 24 inches long, 8 inches wide, and had two bars spaced 8 inches apart (making the marked area 24 inches by 24 inches) were spaced every 100-150 ft and were analyzed along horizontal curves in Arizona by Latoski (2009) in an observational before-after study. No comparison locations were included in the analysis. Curve operating speed data were collected before, immediately after, and three months after installation of the optical speed bars. Vehicle speeds were collected at a location approximately three quarters of a mi downstream of the terminal end of the speed bars for one week at each time period. Speeds were not collected at any other locations.

The findings indicated that the mean speeds and 85th-percentile speeds in the after periods, for both weekdays and weekends, had statistically significant reductions (at the 99-percent significance level). The reductions in speed were the greatest at nighttime (with reductions of up to 5.5 mph) and the lowest during the daytime (as low as 1.3-mph reduction).

The operational effectiveness of zig-zag pavement markings at roadway crossings with multi-use paths was analyzed by Dougald (2010). The analysis involved a 45-mi-long multi-use path in Virginia with over 70 crossings, two of which received the treatment. The zig-zag pattern was intended to indicate to motorists on a two-lane highway that they were approaching a multi-use path crossing. To estimate the effectiveness of the zig-zag markings, speed data were collected before installation, one week after installation, six months after installation, and one year after installation at both treatment locations. Speed data were also collected at two similar locations that did not receive the treatment – these data were collected at the same time as the treatment site data were collected. Motorist attitudinal changes were also assessed through surveys. These surveys targeted motorists, pedestrians, and bicyclists that were familiar with the zig-zag markings.

The findings indicated that the markings increased the awareness of motorists approaching the crossings as evidenced by reduced mean operating speeds. These findings were based on simple comparisons of the descriptive statistics of the before and after data and no statistical analysis was provided. Also, the survey results indicated that awareness was increased and a higher tendency to yield resulted after the markings were installed. Only descriptive statistics were used for the survey results. An interesting finding from the survey was that motorists had limited understanding of the markings and, that motorists, pedestrians, and cyclists were confused as to who had the right-of-way at the crossing locations.

Another pavement marking scheme that has been used in an attempt to reduce vehicle operating speeds approaching horizontal curves on two-lane rural highways has been an on-pavement "sign" with a curve ahead warning arrow and speed advisory. An example of this treatment is shown in figure 2 and figure 3.

Figure 2. Photo. Horizontal curve warning and speed advisory pavement marking (Retting and Farmer 1998).

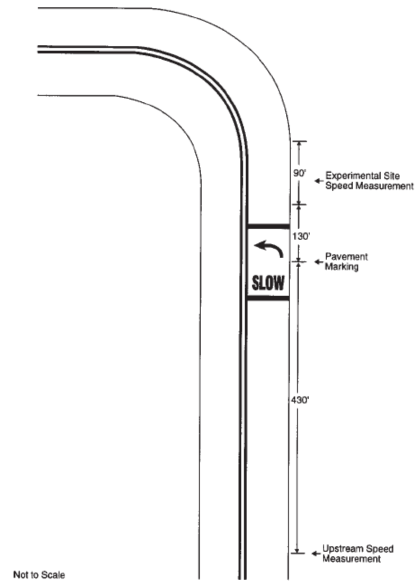

Figure 3. Illustration. Horizontal curve warning and speed advisory pavement marking (Retting and Farmer 1998).

A speed evaluation of this treatment by Retting and Farmer (1998) involved an arrow curved to the left with the word “slow” panted before it at a location approximately 220 ft before the PC of a left-hand horizontal curve. A single curve on a two-lane suburban highway in northern Virginia with paved roadway width of 20 ft received the treatment. A separate curve on the same highway was used as a control. The posted speed limit was 35 mph, while an advisory speed of 15 mph was posted approximately 500 ft before the curve with the treatment.

Vehicle operating speeds were collected at locations 90 ft prior to the PC as well as at a location 650 ft prior to the PC of the treated curve. Speeds at the control curve were collected 100 ft prior to the PC. Speeds were collected five months prior to installation of the pavement markings and two weeks after installation. The results indicated a statistically significant decrease in the mean speed at the location 90 ft prior to the PC, based on a comparison of the curve with the treatment and the control curve in the before and after periods. The magnitude of the reduction was 7 percent (2.4 mph).

A driving simulator was recently used to evaluate the effectiveness of several low-cost treatments on operating speeds for the following two scenarios: 1) two-lane rural horizontal curves at night and 2) traffic calming measures for rural towns during the day (Molino et al., 2010). All drivers partcipating in the study viewed all of the treatment conditions. All drivers were between the ages of 18 and 88 years old (mean age of 57.6 years old). There was an even distribution between male and female drivers. A total of 36 drivers participated in the study.

Treatments that were analyzed for the rural town traffic calming included parked cars along both sides of the road, chicanes, and bulb-outs (curb and gutter or painted types). Treatments for rural horizontal curves included post-mounted delineators (PMDs), light-emitting diode (LED) enhanced PMDs, and edge lines added were there were none previously.

Operating speed and acceleration profiles were collected on the approach to and at the curve treatment locations along rural town road segments. The results of the analysis for the rural town traffic calming treatments indicated that chicanes reduced driver speeds by 6 mph, parked cars on both sides of the road reduced driver speeds by 4 mph, and bulb-outs reduced driver speeds by 1 mph. Results for the horizontal curves indicated that the LED enhanced PMDs were associated with a 9-mph reduction in driver speeds, PMDs were associated with 7-mph reductions in driver speeds, and edge lines were associated with driver speed reductions of 2 mph. All speed reductions were statistically significant.

A simulator was also used by Charlton (2007) to analyze the effects of warning signs on driver speed traversing horizontal curves in New Zealand. The warning sign scenarios included: 1) advance curve warning signs and advisory speeds, 2) advance curve warning signs followed by chevron sight boards, 3) advance curve warning signs followed by chevrons through the curve, and 4) advance curve warning signs followed by chevron sight boards and then chevrons through the curve. Data from 48 drivers were collected, imbalanced between male and female (17 men, 31 women). The findings indicated that the advance warning signs with advisory speeds were ineffective. Further, the results indicated that chevrons produced statistically significant reductions in driver operating speeds. No estimates were reported for the magnitude of the speed reductions.

The effect of chevron signs on vehicle operating speeds and lateral vehicle lane position was investigated by Ré et al. (2010) Two horizontal curves on two-lane rural highways with 10 ft lanes in Texas were used in the evaluation. Both had posted speed limits of 70 mph during the day and 65 mph at night, one had an advisory speed of 45 mph, and the other location had an advisory speed of 50 mph. For the treatment, chevron signs were placed every 160 ft between the PC and point of tangency (PT) of the curves. Two sign configurations were tested -- the first included chevron signs mounted on non-reflective posts, and the second included chevron signs mounted on retroreflective posts.

Speed and lane position data were collected at the PC, midcurve, and 1 mi upstream of the curve (both directions) for each treatment location. The upstream data collection locations were used as control locations. Data were collected before installation of the signs, 10 days after installation of the signs (with retroreflective posts), and 10 days after installation of chevron signs without retroreflective posts. Prior to the analysis, the data were separated into daytime and nighttime observations for each observation period.

The results of the lane position analysis indicated that both types of signs were associated with vehicles moving away from the centerline by approximately 15 inches from the before period to the after period. Additionally, lower variance in lane position was found after the chevron signs were deployed at the treatment sites. The speed data analysis indicated that both types of chevrons produced statistically significant reductions in speeds at the PC and midcurve locations. The magnitude of the reductions in mean operating speeds due to the chevrons was 1.28 mph (for non-retroreflective posts) and 2.2 mph (with retroreflective posts). The 85th-percentile operating speeds were also reduced by 1.3 mph (for non-retroreflective posts) and 2.2 mph (with retroreflective posts). There was not a statistically significant change in the speed variance for any location resulting from implementation of the chevron signs.

The operating speed effects of adding retroreflective posts to existing chevrons and on-pavement curve markings were also evaluated by Hallmark et al. (2012) This study added retroreflective posts to existing chevrons at four horizontal curves and on-pavement curve markings at two horizontal curves (a total of six horizontal curves) on two-lane rural highways in Iowa. Examples of these treatments are shown in figure 4 and figure 5. No comparison locations were used in the evaluation.

Speed data at each of the treatment locations were collected one month before installation, one month after installation, and 12 months after installation of the treatments. Vehicle speeds were collected at the PC, midcurve, and PT for each of the locations. The results indicated that both the added retroreflective posts and on-pavement curve markings were moderately effective in reducing both the mean and 85th-percentile operating speeds both the short-term and long-term. Some of the sites showed little to no change in operating speeds while the majority had statistically significant decreases in the percentage of passenger cars exceeding the speed limit by more than 10 mph (up to 50-percent reduction during the day and 41-percent reduction at night).

Figure 4. Photo. On-pavement marking (Hallmark et al. 2012).

Figure 5. Photo. Chevrons with retroreflective posts (Hallmark et al. 2012).

Effectiveness of Traffic Signs, Pavement Markings, and Other Roadway Treatments in Reducing Roadway Departure Crashes on Horizontal Curves

Table 3 is a summary of the crash modification factors (CMFs) that have been reported in the American Association of State Highway and Traffic Officials (AASHTO) *Highway Safety Manual* (2010) for treatments that may be applied to horizontal curves on two-lane rural highways. These treatments may be considered “proven” as a result of a rigorous, scientific safety evaluation.

A study by Srinivasan et al. (2009) used an empirical Bayes (EB) observational before-after study to determine the safety effects of new chevrons, advance warning signs, horizontal arrows, and improved existing signs using fluorescent yellow sheeting at horizontal curves on two-lane rural roads. Data from Connecticut and Washington were used for the analysis. Horizontal curves in Connecticut received improved signing via improved existing or new signs with fluorescent yellow sheeting. The signs included chevrons, advance warning signs, and horizontal arrows. The road data included 89 treated horizontal curves and 334 horizontal curves in the reference group. Crash data for the years 1995-2006 were used for the analysis. The results indicated statistically significant decreases of 17.8 percent for all crashes, 17.7 percent for lane departure crashes, 35.3 percent for nighttime crashes, and 34.2 percent for nighttime lane departure crashes. CMFs for these estimates are shown in table **4**.

Horizontal curve treatment locations in Washington involved only installation of chevrons where there were none previously or where the number of chevrons was increased. For the analysis, 139 horizontal curves received the treatment and approximately 4,000 horizontal curves were included in the reference group. Crash data for the years 1993-2007, excluding 1997 and 1998, were used for the analysis. Analysis results indicated non-significant reductions in total and lane departure crashes and statistically significant reductions of 24.5 percent for nighttime crashes and 22.1 percent for nighttime lane departure crashes. CMF estimates from this study are shown in table **4**.

Pratt et al. (2014) investigated the effects of pavement surface treatments on safety using data from Texas. A total of 501 horizontal curves on rural highways were included in the evaluation, 101 curves were treated with the HFST, and 400 randomly chosen curves were identified as a reference group. Cross-sectional statistical models were estimated for total, wet-weather, run-off-road, and wet-weather run-off-road crashes for two-lane rural highways. The findings indicate that, as the skid number on a road increases, the expected crash frequency decreases. The benefits of a high skid resistance on a paved surface appeared to be more significant for wet-weather run-off-road and wet-weather crashes, relative to total crashes. The authors' developed a decision-support tool to estimate the potential safety benefit of installing high-friction surface treatments along horizontal curves.

Table 3. CMFs for traffic engineering treatments.

CMF	Area Type	Facility Type	Crash Type	CMF (S.E.)	Source
Install combination horizontal alignment / advisory speed signs	Rural /Suburban	All types	Injury crashes	0.87 (0.09)	Elvik and Vaa (2004)
Install combination horizontal alignment / advisory speed signs	Rural /Suburban	All types	Non-injury crashes	0.71 (0.2)	Elvik and Vaa (2004)
Install changeable speed warning sign	Rural /Suburban	All types Arterials	All types and severities	0.54 (0.2)	Elvik and Vaa (2004)
Place standard edge line markings	Rural	Two-lane	Injury crashes	0.97 (0.04)	Elvik and Vaa (2004)
Place standard edge line markings	Rural	Two-lane	Non-injury crashes	0.97 (0.1)	Elvik and Vaa (2004)
Place wide (8 in.) edge line markings	Rural	Two-lane	Injury crashes	1.05 (0.08)	Elvik and Vaa (2004)
Place wide (8 in.) edge line markings	Rural	Two-lane	Non-injury crashes	0.99 (0.2)	Elvik and Vaa (2004)
Place centerline pavement markings	Rural	Two-lane	Injury crashes	0.99 (0.06)	Elvik and Vaa (2004)
Place centerline pavement markings	Rural	Two-lane	Non-injury crashes	1.01 (0.05)	Elvik and Vaa (2004)
Place center and edge line markings	Rural	Two / Multilane	Injury crashes	0.76 (0.1)	Elvik and Vaa (2004)
Install edge lines, centerlines, PMDs	Rural	Two / Multilane	Injury crashes	0.55 (0.1)	Elvik and Vaa (2004)
Install snowplowable, permanent RPMs	Rural	Two-lane and freeways	Nighttime crashes	0.67–1.43 (depending on AADT)	Elvik and Vaa (2004)
Install centerline rumble strips	Rural	Two-lane	All crashes	0.86 (0.05) 0.91 (0.04)	Persaud et al. (2003) & Torbic et al. (2009)
Install centerline rumble strips	Rural	Two-lane	Fatal and injury	0.88 (0.03)	Torbic et al. (2009)
Install centerline rumble strips	Rural	Two-lane	All injury crashes	0.85 (0.08)	Persaud et al. (2003)
Install centerline rumble strips	Rural	Two-lane	Head-on and sideswipe crashes	0.79 (0.1)	Persaud et al. (2003)
Install centerline rumble strips	Rural	Two-lane	Head-on and sideswipe injury crashes	0.75 (0.2)	Persaud et al. (2003)
Install automated speed enforcement	All types	All types	Injury crashes	0.83 (0.01)	AASHTO (2010)
Install changeable speed warning signs for individual drivers	All types	All types	All crashes	0.54 (0.2)	Elvik and Vaa (2004)

Note: SVROR = Single-vehicle run-off-road.

Table 4. Select horizontal curve CMFs.

Treatment	Roadway Type	Crash Types	Expected CMF (S.E.)	Source
Improve signs with fluorescent sheeting	Two-lane rural highway	All	0.822 (0.077)	Srinivasan et al. (2009)
Improve signs with fluorescent sheeting	Two-lane rural highway	Run-off-road	0.823 (0.084)	Srinivasan et al. (2009)
Improve signs with fluorescent sheeting	Two-lane rural highway	Nighttime	0.647 (0.105)	Srinivasan et al. (2009)
Improve signs with fluorescent sheeting	Two-lane rural highway	Nighttime run off road	0.658 (0.115)	Srinivasan et al. (2009)
Add chevrons	Two-lane rural highway	All	0.957 (0.089)	Srinivasan et al. (2009)
Add chevrons	Two-lane rural highway	Run off road	0.941 (0.088)	Srinivasan et al. (2009)
Add chevrons	Two-lane rural highway	Nighttime	0.755 (0.127)	Srinivasan et al. (2009)
Add chevrons	Two-lane rural highway	Nighttime run off road	0.779 (0.101)	Srinivasan et al. (2009)

Effects of Roadway and Roadside Features on Safety and Driver Speed Choice

Safety

Similar to the CMFs shown in table 3, the *Highway Safety Manual* (HSM) includes a collection of CMFs for roadway and roadside design features on rural two-lane highways that may influence roadway departure crashes. (AASHTO 2010) These are presented in table 5. The CMFs for lane width, shoulder width and type, and roadside hazard rating in table 5 apply to SVROR and opposite-direction crashes; however, algorithms are included in the HSM to convert these target crash CMFs to total crashes.

Similarly, in the NCHRP Report 500 Volume 6, Neuman et al. (2003) provided suggested treatments for reducing run-off-road crashes. Treatments were separated into categories by the objective of the countermeasures. Categories included: 1) keeping vehicles from encroaching into the roadside, 2) minimizing the likelihood of crashing into an object or overturning if the vehicle travels beyond the edge of the shoulder, and 3) reduce the severity of the crash. Of the suggested countermeasures, there are tried (implemented and may even be accepted as standards, but there are no valid evaluations of their effectiveness), experimental (no properly designed studies have been done and have not been implemented and accepted as a standard), and proven (properly designed evaluations have found to be effective).

Suggested treatments for keeping vehicles from encroaching into the roadside, with ratings according to the report, included: 1) installing shoulder rumble strips [tried], 2) installing edge line rumble strips or edge line profile marking on road sections with narrow or no paved shoulders [experimental], 3) install center lane rumble strips [experimental], 4) provide enhanced shoulder or in-lane delineation and marking for sharp curves [proven, tried, and experimental], 5) provide improved highway geometry for horizontal curves [proven], 6) provide enhanced pavement markings [tried], 7) provide skid-resistant pavement surfaces [tried], 8) eliminate shoulder drop-offs [experimental], and 9) widen and/or pave shoulders [proven]. Treatments suggested to minimize the likelihood of crashing into an object or overturning if the vehicle travels beyond the edge of the shoulder included: 1) designing safer side slopes and ditches to prevent rollovers [proven], 2) remove or relocate objects in hazardous locations [proven], and 3) delineate trees or utility poles with retroreflective tape [experimental]. Finally, in order to reduce the severity of crashes, the report suggested either improving the design of roadside hardware [tried] or improving the design and application of barrier and attenuation systems [tried].

Table 5. CMFs for roadway and roadside features.

CMF	Area Type	Facility Type	Notes	CMF (S.E.)
Modify lane width	Rural	Two-lane	None	See figure 13-1 from HSM
Add or widen paved shoulder	Rural	Two-lane	None	See figure 13-5 from HSM
Increase the distance to roadside features	Rural	Two-lane and freeways	3.3 ft–16.7 ft offset	0.78 (0.02)
Increase the distance to roadside features	Rural	Two-lane and freeways	16.7 ft–30.0 ft offset	0.56 (0.01)
Reduce roadside hazard rating	Rural	Two-lane	$CMF = \frac{e^{(-0.6869+0.0668 \times RHR)}}{e^{-0.4865}}$	See figure 13-8 from HSM
Modify horizontal curve radius	Rural	Two-lane	$CMF = \frac{(1.55L_c)+\left(\frac{80.2}{R}\right)-(0.012S)}{1.55L_c}$	See figure 13-9 from HSM
Improve superelevation	Rural	Two-lane	Improve superelevation variance (SV) < 0.01	1.00 (N/A)
Improve superelevation	Rural	Two-lane	Improve SV 0.01 ≤ SV < 0.02	1.00 + 6 (SV – 0.01)
Improve superelevation	Rural	Two-lane	Improve SV > 0.02	1.06 + 3 (SV – 0.02)
Change vertical grade	Rural	Two-lane	Increase vertical grade by 1 % (SVROR)	1.04 (0.02)

Gross et al. (2009) used a case-control study to analyze the effect of lane width – shoulder width combinations on crash frequency. Data for the study came from Pennsylvania and Washington States and consisted of two-lane rural road segments (intersections were not included in the analysis). Roadway and crash data covered five years for Pennsylvania and six years for Washington. Cases for this study were defined as road segments that had at least one reported crash during one analysis year. Controls were taken from road segments that did not have any reported crashes during the same year. The final recommended CMFs are shown in figure 6, and are relative to a 36-ft baseline with 12-ft travel lanes and 6-ft shoulders. As shown, 34 ft total width with either 11 or 12 ft lanes lower crash frequency compared to the baseline, 36 ft total width with 11 ft lanes are the same, and all other combinations result in higher crash frequencies than the baseline.

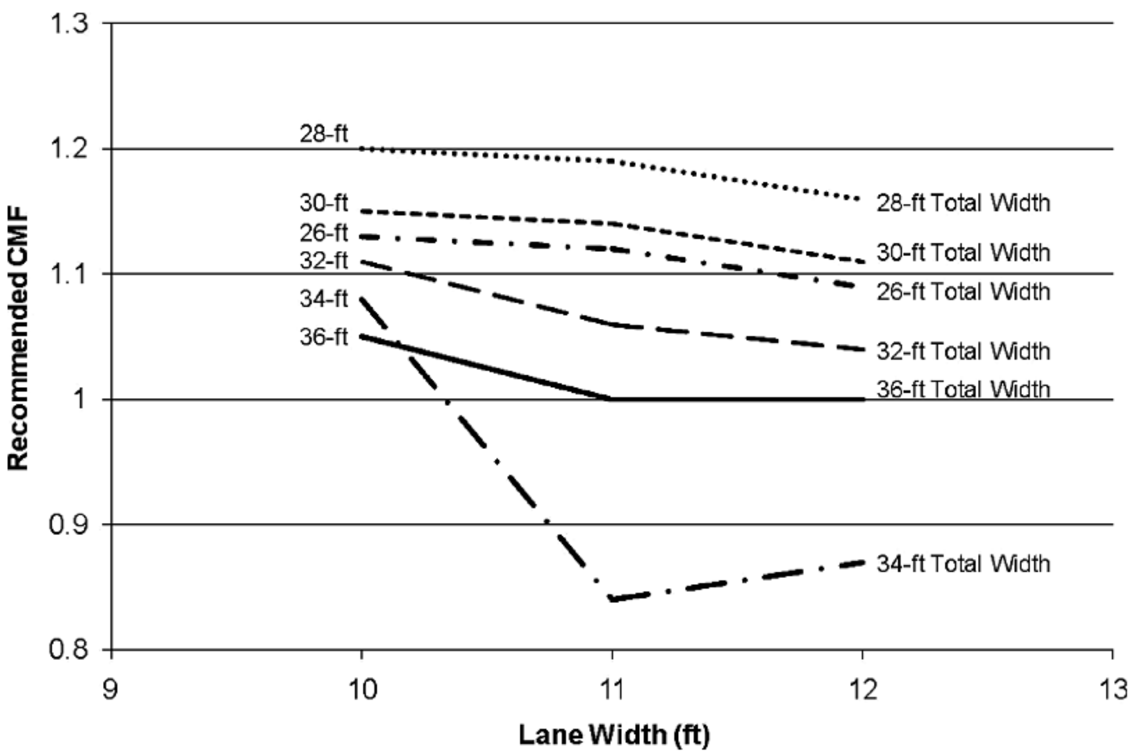

Figure 6. Graph. Recommended CMFs using lane width and shoulder width combinations (Gross et al. 2009).

Centerline rumble strips on two-lane rural roads were evaluated for potential safety benefits by Persaud et al. (2004) An EB before-after methodology was employed to analyze roadway and crash data from California, Colorado, Delaware, Maryland, Minnesota, Oregon, and Washington. A total of 98 treated curves along 210 mi of roadway were used. Highway Safety Information System (HSIS) crash and roadway inventory data were used to calibrate the safety performance functions (SPFs) which were used for the analysis. The results of the analysis indicated that, for all States combined, centerline rumble strips resulted in a statistically significant reduction in crashes. Results for individual States varied from significant reductions (all crash types in California, Colorado, and Washington, opposing direction in Colorado and Minnesota) to insignificant reductions, no change, or an insignificant increase in crashes for all other locations and crash types. Composite results indicated a 12-percent reduction in all crashes, 25-percent reduction in opposing direction crashes, 15-percent reduction in total crashes at night, and 8-

percent reduction in total crashes during the day. CMFs from the composite results are shown in table 6.

Park et al. (2012) estimated the safety effects of wider edge lines on two-lane rural roads using data from Kansas, Michigan, and Iowa were used for analysis. Analysis methods were determined based on the data available for each State and included an EB before-after study (Kansas), interrupted time series design and generalized linear segmented regression (Michigan), and cross-sectional analysis (Illinois). All road data were non-intersection, non-interchange segments and crash data were for non-winter months only.

For Kansas, data included 2,801 road segments with crashes for the years 2001-2008. Treated segments received wider painted edge lines (change from 4 inches to 6 inches) during the years 2005-2007. Of the total segments, 263 were used to develop the SPFs (i.e., reference group) and the rest received the treatment and were used for estimation of the safety effects of the wider edge lines. Results from the Kansas data analysis indicated a 17.5-percent reduction in total crashes, 28.6-percent reduction in daytime crashes, 30.2-percent reduction in single vehicle crashes, and 18.5-percent reduction in fixed object crashes. All of the reported crash reductions were statistically significant. CMFs from the estimates are shown in table 6.

Data from Michigan included 253 total road segments with crash data for the years 2001-2009. For Michigan, edge lines for almost all State-owned roads changed from 4 inches to 6 inches in width in 2004. Therefore, almost no comparable roads for 2004-2009 were available for use as a comparison or reference group. Interrupted time series analysis of the data indicated a statistically significant reduction of 19.4 percent for all crashes, 18.8-percent reduction in nighttime crashes, and 18.7-percent reduction in single-vehicle crashes.

Crash data for 6,531 road segments for the period 2001-2006 were used for the Illinois analysis. From these segments, 5,343 had 4 inch centerlines and 4 inch edge lines. The remaining 1,188 segments had centerlines and edge lines that were 5 inches wide. Before the analysis was conducted, all animal-related crashes were removed from the data files (approximately 50 percent of the crashes). The analysis results indicated a 30.1-percent reduction in total crashes, 29.1-percent reduction in daytime crashes, 29.9-percent reduction in nighttime crashes, 37.0-percent reduction in single-vehicle crashes, and 29.5-percent reduction in fixed object crashes. CMFs for these crash reductions are shown in table 6.

Table 6. Select rural two-lane highway CMFs.

Treatment	Crash Types	CMF (S.E.)	Significant
Centerline rumble strips	All	0.86 (0.03)	Yes
Centerline rumble strips	Opposing direction	0.75 (0.087)	Yes
Centerline rumble strips	Daytime	0.92 (0.048)	Yes
Centerline rumble strips	Nighttime	0.85 (0.028)	Yes
Widen edge lines (from 4 in to 6 in)	All	0.825 (0.028)	Yes
Widen edge lines (from 4 in to 6 in)	Daytime	0.714 (0.043)	Yes
Widen edge lines (from 4 in to 6 in)	Single vehicle	0.698 (0.044)	Yes
Widen edge lines (from 4 in to 6 in)	Fixed object	0.815 (0.066)	Yes
Widen edge lines (from 4 in to 5 in)	All	0.699 (0.043)	Yes
Widen edge lines (from 4 in to 5 in)	Daytime	0.709 (0.052)	Yes
Widen edge lines (from 4 in to 5 in)	Nighttime	0.701 (0.07)	Yes
Widen edge lines (from 4 in to 5 in)	Single vehicle	0.630 (0.048)	Yes
Widen edge lines (from 4 in to 5 in)	Fixed object	0.705 (0.064)	Yes

The impact of pavement conditions on crash severity at horizontal curves was investigated by Buddhavarapu et al. (2012) The study used crash data from an unspecified number of horizontal curves on two-lane highways in Texas, along with pavement condition (skid index, distress index, and International Roughness Index) and other roadway characteristic data for the analysis. The skid index was a measure of longitudinal surface friction. The distress index served as a measure of the extent of surface distress on the pavement (such as cracking, edge drop offs, rutting, and raveling). The International Roughness Index (IRI) is a measure of the roughness of longitudinal road profiles (higher IRI values indicate lower ride quality). Data for the years 2006-2009 were used for statistical modeling.

The modeling results indicated that the longitudinal skid index did not have a statistically significant effect on crash severity. Further, the results indicated that the distress index and IRI were both statistically significant with positive and negative signs, respectively. The interpretation of the parameter estimate for distress index was that the probability of a crash being more severe is greater at horizontal curves with less distressed pavements than at horizontal curves with more distressed pavement. Also, the interpretation of the parameter estimate for IRI is that as the ride quality decreases (less smooth ride), the probability of crashes being more severe is reduced.

Speed Choice

Edge line treatments were analyzed to determine the effect on operating speeds along rural two-lane highways by Tsyganov et al. (2006) This study evaluated differences between road segments with and without road edge lines in Texas. The research methods included laboratory testing of the effects of edge lines on vehicle speeds and lane position for various lane widths using t-tests. Findings indicated that edge lines were associated with speed increases of approximately 5 mph. However, this increase was not statistically significant. Findings for lane position indicated that there were no significant differences in the lane position of vehicles resulting from edge line applications.

Other roadway and roadside features and characteristics along or near horizontal curves that have been researched for impacts on speed choice include curve radii, degree of curvature, grade, length of curve, posted speed limit, available sight distance, paved width, gravel shoulder width, clear zone, superelevation, land use, roadside hazard rating, crest vertical curves, curvature change rate, and deflection angle. Each of these features/characteristics have been used in regression models to predict operating speed parameters. Table 7 shows the operating speed parameter that was modeled (dependent variable) in the first column, the independent variables (the aforementioned features/characteristics) used in the respective models in the second column, the country the data used came from in the third column, and references are shown in the last column.

Table 7. Operating speed models.

Model	Predictors	Country	Reference
V_{85}	Radius	USA	Taragin (1954)
V_{85}	Radius	USA	Glennon et al. (1986)
V_{85}	Degree of curvature	USA	Islam & Seneviratne (1994)
V_{85}	Radius	USA	Voigt (1996)
V_{85}	Radius	USA	Passetti & Fambro (1999)
V_{85}	Radius, K, grade	USA	Fitzpatrick et al. (2000)
V_{85}	Degree of curvature, length of curve	USA	Ottesen & Krammes (2000)
$\mu_{V,Tangent}$, $\sigma_{V,Tangent}$	Percentage trucks, posted speed, grade, residential land use, sight distance, intersections, paved width, gravel shoulder width, untreated shoulder width, clear zone, flat curve (radius > 1,700 ft)	USA	Medina & Tarko (2005)
$\mu_{V,Curve}$, $\sigma_{V,Curve}$	Sight distance, residential land use, superelevation	USA	Medina & Tarko (2005)
Deceleration Rate	Left-hand curve, radius, approach tangent length, curve length, roadside hazard rating	USA	Hu & Donnell (2010)
Acceleration Rate	Left-hand curve, radius, approach tangent length, curve length, roadside hazard rating, departure tangent length	USA	Hu & Donnell (2010)
V_{Posted}, μ_V, σ_V	Access points, traffic volume, residential land use, industrial land use, curb and gutter, grade, on-street parking, median or turning lane, shoulder width, at-grade rail crossing, left-hand curve, crest curve, wooded adjacent land, percentage trucks	USA	Himes et al. (2011)
$V_{85,Tangent}$, $V_{85,Curve}$	Radius, length of curve, preceding tangent length	India	Jacob & Anjaneyulu (2012)
μ_V, σ_V	Radius, length of curve	India	Jacob & Anjaneyulu (2012)
ΔV_{85}	Radius, length of curve, tangent velocity	India	Jacob & Anjaneyulu (2012)
V_{85}	μ_V	Germany	Koeppel (1984)
μ_V	Pavement width, curvature change rate	Germany	Koeppel (1984)

V_{85}	Radius, pavement width	Germany	Lippold (1997)
V_{85}	Curvature change rate, V_{env}	Italy	Cafiso et al (2008)
V_{env}	Pavement width, curvature change rate	Italy	Cafiso et al (2008)
V_{85}	Radius, tangent length preceding the curve, deflection angle	UK	Bird & Hashim (2005)
V_{85}	Radius	Greece	Kanellaidis et al. (2000)
V_{85}	Curvature change rate	Greece	Xenakis (2008)

Note: V_{85} = 85th percentile operating speed, μ_V = mean operating speed, $\sigma_{V,}$ = speed variance, V_{Posted} = posted speed limit, ΔV_{85} = the change in 85th percentile operating speed, and V_{env} = environmental speed.

Applications of Speed-Activated Traffic Control Devices on Horizontal Curves and Other Locations

Hallmark et al. (2013) analyzed the impacts of two different dynamic speed feedback signs (DSFS) on operating speeds of two lane rural roads at locations with sharp horizontal curves. An example of dynamic speed display signs (DSDS) is shown in figure 7. A total of 22 locations were used, with locations in Arizona, Florida, Iowa, Ohio, Oregon, Texas, and Washington. The sites were selected based on several criteria. First, the locations had to have a high number of crashes and no adverse geometric conditions (such as a major intersection or railroad crossing). The sites also had to have known speeding problems (defined as free flow vehicle speed distributions with the 85th percentile speed being 5 or more mph greater than the posted speed). The final locations (from a pool of sites that met all criteria) were selected based on the number and type of crashes, speeding conditions, and practicality of installing the signs at the locations.

The two different DSFS that were evaluated were 1) a sign that displayed the vehicles speed (a speed display sign), and 2) a curve display sign. The curve display signs were programmed to only turn on and display a message if it detected a vehicle exceeding a set threshold. The speed display signs were also activated when they detected vehicles exceeding a set threshold. The speed display signs would then show the vehicles speed unless the vehicle was traveling at 20 or more mph greater than the speed limit, in which case it would display the actual posted speed. This was done to discourage drivers from using the speed display sign to test their speeds.

Speed measurements were conducted at each site using pneumatic tubes and counters before installation as well as 1 month, 12 months, and 24 months after installation. Data collection lasted for 48 hours per site per period, when possible, and occurred on weekdays. The collection of speeds 24 months after installation was used to determine if the speed reductions lasted over time, if there were any to begin with.

Summary statistics indicated that speed reductions due to the installation of the signs ranged from 0.6 mph to 6.5 mph in the mean speeds and 1 mph to 8 mph in the 85th-percentile speeds at the PC. At the midcurve, speed changes for the mean speed ranged from an increase of 3.7 mph to a decrease of 7.9 mph. The change in speeds for the 85th-percentile at the midcurve ranged from an increase of 3 mph to a decrease of 9 mph. It was also noted that problems encountered during the study period included one sign stopped functioning, software issues, wiring replacement issues, needs to realign or replace solar panels, replacement of fuses, and issues with vandalism.

Figure 7. Photo. DSDS sign. (Hallmark et al. 2013).

A subsequent report by Hallmark et al. (2015) described a safety evaluation of the 22 DSDS sites that were included in the operating speed evaluation reported above. In this evaluation, 29 control site locations (DSDS was not deployed at these curve locations) were included. Crash data were collected for up to 4 years before and up to 3 years after implementing the signs. The results showed that the CMFs for total crashes was 0.95 (with 0.01 standard error) for both directions of travel, and 0.93 (with 0.02 standard error) for the direction in which the sign was present. The single-vehicle crash CMF was 0.95 (with 0.01 standard error) for both directions of travel, and 0.95 (with 0.02 standard error) for the direction toward the sign. All the CMFs were statistically significant.

An analysis of DSDS installed in several permanent locations, including two "sharp horizontal curves" was performed by Jeihani et al. (2012) The analysis looked at the impacts of DSDS on vehicle speeds. Speed data for this study were collected before installations of DSDS, approximately one week after installation, and approximately four months after installation. All speed data were collected using hand-held lidar guns at a control point upstream of the locations as well as adjacent to the location of the DSDS. The collection one week after installation was used to estimate the initial effect of DSDS. The collection four months after DSDS installation was used to determine how well the initial impacts on speeds were maintained. Summary statistics as well as simple linear regression were used to analyze the data. The summary statistics indicated that for the two sharp curves, in the speeds collected one week after the installation of the signs, the speed changes ranged from 3.5-mph reduction to an increase of 0.6 mph for the mean auto speeds and 2.4-mph decrease to an increase of 1.3 mph for trucks mean speeds. For the 85^{th}-percentile speeds, the speed changes for passenger cars ranged from 4-mph decrease to no change and speeds changes for trucks ranged from 3-mph decrease to no change. Also, for the sharp horizontal curves, the speed data was disaggregated to develop regression models for cars and large trucks separately at each time point and location. Findings indicated that the car model for one of the curves at four months after installation was significantly different from the before period and one week after period and that vehicle speeds had been reduced. All other models for the sharp curves were not statistically different.

Tribbett et al. (2000) analyzed the effectiveness of dynamic curve warning signs installed by CALTRANS at five separate locations along Interstate 5 in the Sacramento River Canyon. Interstate 5 in Sacramento River Canyon is a rural, winding, four-lane freeway (two lanes in each direction). The purpose of the signs was to provide advance notice to motorists of alignment changes and speed advisories. The signs were able to display both text and diagrammatic warnings. The signs were equipped with radar technology, so the speed of approaching vehicles could be identified. Based on the detected speeds, the display message would change.

This project investigated the impacts of dynamic curve warning signs on crash frequency, erratic maneuvers, operating speeds, public acceptance, public response, and maintenance requirements. The operating speeds were collected at nine months prior to installation of the signs, shortly after installation, three months after installation, and eight months after installation.

The immediate impacts of the dynamic curve warning signs on operating speeds were a reduction in speed. Over time, vehicle operating speeds increased until there was no statistically significant difference between the speeds before and after installation of the signs. The authors noted that curves on steep grades seemed to have the largest speed reductions. For the crash analysis, the findings indicated that crash frequency was reduced. Erratic behaviors were the same in the before and after periods for all but one of the locations. The one location with a difference in erratic behavior indicated that erratic behavior was reduced in the after period. Public response and acceptance were reported as favorable. There were reportedly few maintenance difficulties with the signs.

DSDS were also analyzed for speed reductions in transitions zones of two-lane highways in Pennsylvania by Cruzado and Donnell (2009). Data were collected at 12 transition zones at locations before implementation, during the first week of DSDS use, and one week after the DSDS was removed from the transition zone. Speeds were collected at positions one-half mi prior to the DSDS, at the DSDS location, and 500 ft downstream of the DSDS location. Only the speeds of free-flow vehicles were included in the analysis. The results indicated average reductions in operating speeds of 6 mph while DSDS was in use. The results of the analysis of the data after the DSDS was removed indicated that the benefits faded after the DSDS devices were removed.

Further research into DSDS impacts on operating speeds was undertaken by Ullman and Rose (2005). This project investigated the impacts of DSDS at various locations including a school speed zone, two sharp horizontal curves, and two approached to signalized intersections on high-speed roadways (all in Texas). Speed data for each location were collected before installation, 1-3 weeks after installation, and 2-4 months after installation of the DSDS. Speed sensors were placed at locations 2,000-3,000 ft upstream of the DSDS as well as near the DSDS. The authors' reported that the DSDS could yield 1-to 4-mph reductions in the average and 85th-percentile operating speeds, but that over time the effect diminishes.

SUMMARY OF LITERATURE REVIEW FINDINGS

Existing literature on treatments and characteristics that have been found to impact safety, operating speed, or both, along rural horizontal curves or two-lane rural highways were identified and summarized in the literature review.

Numerous treatments were evaluated by different researchers attempting to find treatments that reduce operating speeds at horizontal curves. Treatments that were found to have an impact on operating speeds included optical speed bars, curve arrows painted on the road with the word "slow" painted beneath them, post mounted delineators, LED enhanced post mounted delineators, adding edge lines, chevrons, and chevrons with retroreflective posts. Magnitudes of reductions attributed to the treatments ranged from being too small for practical purposes (0.05 mph for optical speed bars) to 9 mph for LED enhanced post mounted delineators.

Various traffic signs, pavement markings, and other treatments were also analyzed by researchers for their effectiveness in reducing crash frequency at horizontal curves. Treatments that were found to be effective included advisory speeds, chevrons, horizontal arrows, improving existing signs using fluorescent yellow sheeting, and sequential flashing beacons. CMFs from these treatments ranged from 0.524 for all crash types on freeways or expressways using a combination of chevron signs, curve warning signs, and flashing beacons, to 0.974 for all crash types when using chevron signs on freeways or expressways. For two-lane rural highways, CMFs ranged from 0.647 for nighttime crashes by improving existing signs with yellow fluorescent sheeting to 0.957 for all crash types when adding chevrons.

Other roadway treatments that have been found to influence safety on two-lane rural highways include modifying lane width, modifying shoulder width, changing the lane width and shoulder in combination, pavement resurfacing, improving signing, repainting pavement markings, adding centerline rumble strips, and widening edge lines. CMFs for these treatments ranged from 0.64 for wet weather crashes after pavement resurfacing project to 1.02 for all crash types for pavement resurfacing projects.

Impacts of edge lines on operating speed were analyzed and found to be associated with a 5-mph reduction on operating speeds, but the decrease was not statistically significant. Other factors have been shown to influence operating speeds on two-lane highways, including curve radii, degree of curve, grade, length of horizontal curve, posted speed limit, available sight distance, paved roadway width, gravel shoulder width, clear zone, superelevation, land use, roadside hazard rating, crest curves, curvature change rate, and deflection angle.

Research analyzing the impacts of various speed activated traffic control devices had varying results on operating speeds. DSDS were found to be associated with a statistically significant reduction in operating speeds of 1-4 mph in short term applications, but that the effect dissipated over time.

CHAPTER 3. OVERVIEW OF SAFETY EVALUATIONS

Based on the findings from the literature review, there are countermeasures being used to mitigate roadway departure crashes on two-lane rural highways that have not been extensively evaluated. The purpose of the remaining chapters of this report are to document efforts that were undertaken in the present study to evaluate three roadway treatment or design practices that focus on "preventing the vehicle from the leaving the roadway." The three evaluations reported in subsequent chapters are as follows:

- Chapters 4 and 5: Safety-Design Consistency Relationship – this evaluation was completed in two parts. The first part considered the safety effects of geometric design consistency using data from Utah and Washington. A subsequent update to the original evaluation used the Strategic Highway Research Program (SHRP) 2 Roadway Inventory Database (RID) to estimate cross-sectional models of expected crash frequency as a function of geometric design consistency variables. The study used high-accuracy roadway data collected explicitly for research purposes to consistency the effects of design decision-making on roadway departure crashes.
- Chapter 6: Guiderail with Post-Mounted Delineators – this evaluation also used the SHRP 2 RID information to estimate the safety effects of guiderail with delineators, guiderail only, and delineators only placed along horizontal curves of two-lane rural highways. A propensity scores-potential outcomes framework was used to estimate crash modification factors (CMFs) for each treatment type on total crashes, fatal plus injury crashes, run-off-road crashes, and nighttime crashes.
- Chapter 7: Horizontal Curve Pavement Marking Warnings – an observational before-after safety evaluation, using the empirical Bayes (EB) method, was completed to evaluate the safety performance of horizontal curve warning pavement markings on two-lane rural highways. CMFs for total, fatal plus injury, and run-off-road crashes were developed.

CHAPTER 4. ROADWAY DEPARTURE SAFETY EFFECTS OF DESIGN CONSISTENCY

Design consistency in the context of highway and street design refers to the conformance of highway geometry to driver expectancy. Various design practices are used to achieve design consistency. Highway designers, for example, seek to make design decisions that are consistent with previous experience (e.g., exit ramp on right). Designers also coordinate horizontal and vertical alignments. Several alternatives for quantifying design consistency on rural, two-lane highways were explored during previous Federal Highway Administration (FHWA)-sponsored research, including operating speed profiles and alignment indices. Operating speed profiles are currently used in the Design Consistency Module of the Interactive Highway Safety Design Model (IHSDM) to "flag" different levels of rural, two-lane design consistency.

While design consistency has safety implications and is intuitively linked to roadway departure crashes, only a few studies have been published that sought to link measures of design consistency to safety (Fitzpatrick et al., 2000; Ng & Sayed, 2004; Wu et al., 2013). These studies offered a starting point for additional analysis, but did not necessarily provide generalizable safety findings related to roadway departure crashes on horizontal curves along rural, two-lane roads in the U.S. The availability of detailed horizontal and vertical alignment data in Utah and Washington offered the potential to explore this relationship at a greater level of breadth and depth.

RESEARCH OBJECTIVE

The objective of this research was to explore and estimate relationships between alternative measures of design consistency and the expected number of roadway departure crashes along horizontal curves on rural, two-lane, two-way roads.

METHODOLOGY

The relationships between alternative measures of design consistency and the expected number of roadway departure crashes were explored using a negative binomial (NB) regression modeling approach. Figure 8 illustrates the general form of an NB regression model of the expected number of roadway departure crashes along horizontal curve "i."

$$E(Y_i) = \mu_i = EXP(X_i\beta+\varepsilon_i)$$

Figure 8. Equation. General form of NB regression model.

where: Y_i = a random variable representing the number of roadway departure crashes along the th horizontal curve;

$E(Y_i) = \mu_i$ = expected number of roadway departure crashes along the th horizontal curve;

X_i = a matrix of explanatory variables characterizing the th horizontal curve and associated with λ_i, including measures of design consistency;

β = a vector of parameters to be estimated that quantify the relationships between the explanatory variables and μ_i; and

ε_i = a disturbance term, where EXP(ε_i) is gamma-distributed with a mean equal to one and variance equal to α_i.

NB regression models include the specification of "additional dispersion" (additional when compared to a Poisson model) that is common to crash data. Statistical road safety modelers often specify this additional dispersion as a dispersion parameter multiplied by the expected number of crashes squared (NB-2 model) and treat the dispersion parameter as fixed across observations. In the NB-2 model with a fixed dispersion parameter (α), the variance in the number of crashes is written as $Var(Y_i) = \mu_i + \alpha\mu_i^2$.

Model estimation for this analysis was carried out using maximum likelihood as implemented in Stata 14.0.

DATA

Road Segments

The road segments used for analysis and modeling each consisted of one horizontal curve (i.e., each observation was defined by the observed number of roadway departure crashes occurring on an individual horizontal curve during the observation period). The analysis also considered characteristics of the "upstream" horizontal tangent, "upstream" horizontal curve, "downstream" horizontal tangent, and "downstream" horizontal curve. Figure 9 illustrates these elements of the analysis. The final database consisted of 249 observations from Washington State and 421 observations from Utah. Data on the starting and ending locations of each horizontal curve and tangent was drawn from databases developed for a concurrent FHWA "Predisposing Factors" project. (Porter et al. 2018)

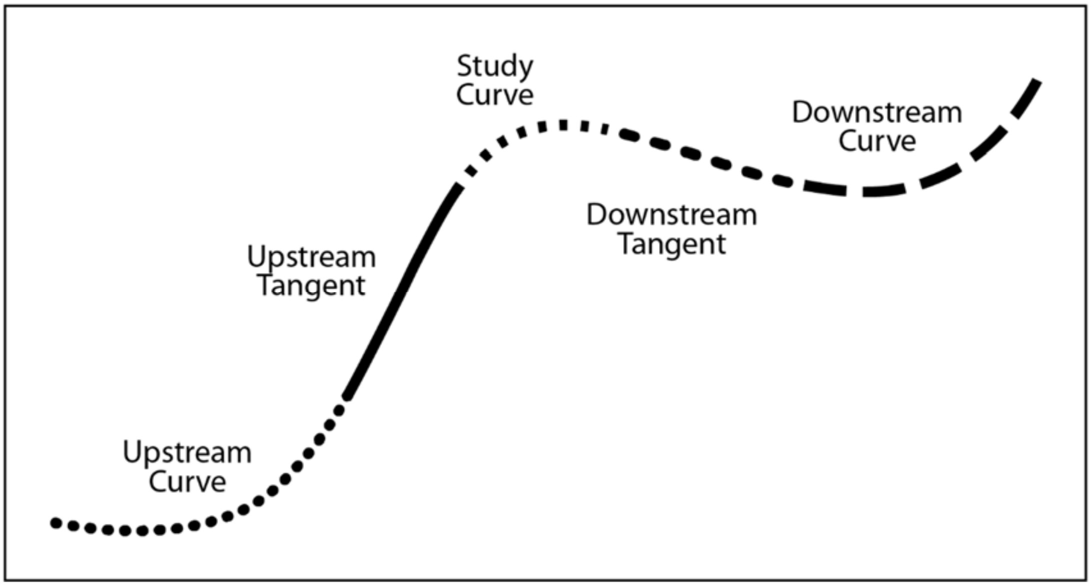

Figure 9. Graphic. Illustration of analysis curve, upstream elements, and downstream elements.

Roadway Data

Traffic and roadway characteristics corresponding to each analysis curve as well as the upstream and downstream tangents and curves were also drawn from databases developed for the concurrent FHWA "Predisposing Factors" project. Some additional calculation steps specific to this analysis were needed in some cases.

The elements in the FHWA "Predisposing Factors" project databases that were of interest to this analysis came from the Highway Safety Information System (HSIS) for Washington State and from the Utah Department of Transportation (UDOT) Data Portal for Utah, with two exceptions. UDOT staff members overseeing UDOT's LiDAR data efforts did not post information on horizontal curvature and superelevation, knowing that data for these elements needed additional processing and quality control. The processing and quality control for horizontal curvature occurred primarily during the concurrent FHWA "Predisposing Factors" project. Processing and quality control for superelevation took place as part of this effort and is described in the following section of this report.

Superelevation

Cross slope measurements at a frequency of 0.1 mi were available in UDOT's processed LiDAR data files. "Noise" in the cross slope measurements became immediately evident. Multiple measurements for a given curve, made in either in the same or opposite travel directions at locations all likely to be at "full superelevation," were different. That said, cross slope measurements generally looked reasonable for given radius-posted speed limit combinations (information on design speed was not available).

Several alternative "simple" data smoothing approaches were tested to arrive at an estimate of design superelevation rate for each horizontal curve:

1. <u>Alternative 1:</u> Take the averages of all cross-slope measurements made along the horizontal curve separately for both the increasing milepost and decreasing milepost directions. Select the average cross slope (i.e., increasing milepost or decreasing milepost) that is closest to the expected design superelevation rate assuming the posted speed limit equals the design speed, maximum superelevation equals 6 percent, and superelevation and side friction are distributed according Method 5 (with values drawn from the 2011 version of *A Policy on Geometric Design of Highways and Streets*).
2. <u>Alternative 2:</u> Take the averages of all cross-slope measurements made between the one-quarter point and three-quarter point of the horizontal curve separately for both the increasing milepost and decreasing milepost directions. Select the average cross slope (i.e., increasing milepost or decreasing milepost) that is closest to the expected design superelevation rate assuming the posted speed limit equals the design speed, maximum superelevation equals 6 percent, and superelevation and side friction are distributed according Method 5 (with values drawn from the 2011 version of *A Policy on Geometric Design of Highways and Streets*).
3. <u>Alternative 3:</u> Take one overall average of all cross-slope measurements made between the one-quarter point and three-quarter point of the horizontal curve in both the increasing milepost and decreasing milepost directions.
4. <u>Alternative 4:</u> Take one overall average of all cross-slope measurements made along the horizontal curve in both the increasing milepost and decreasing milepost directions.

The four alternative superelevation rate estimates were then compared to the expected design superelevation rate assuming the posted speed limit equals the design speed, maximum superelevation equals 6 percent, and superelevation and side friction are distributed according Method 5 (with values drawn from the 2011 version of *A Policy on Geometric Design of Highways and Streets*). Figure 10 contains the results of these comparisons. The figure shows, for example, that for Alternative 1:

- 57.8 percent of the superelevation estimates are within 1 percent of the expected design superelevation rate.
- 77.5 percent of the superelevation estimates are within 1.5 percent of the expected design superelevation rate.
- 88.0 percent of the superelevation estimates are within 2 percent of the expected design superelevation rate
- 94.7 percent of the superelevation estimates are within 2.5 percent of the expected design superelevation rate.

The research team concluded that Alternative 3 likely produces the most robust superelevation estimates since it utilizes measurements from both travel directions at locations where the cross slope is at full superelevation. Estimates from Alternative 3 were used during the statistical modeling.

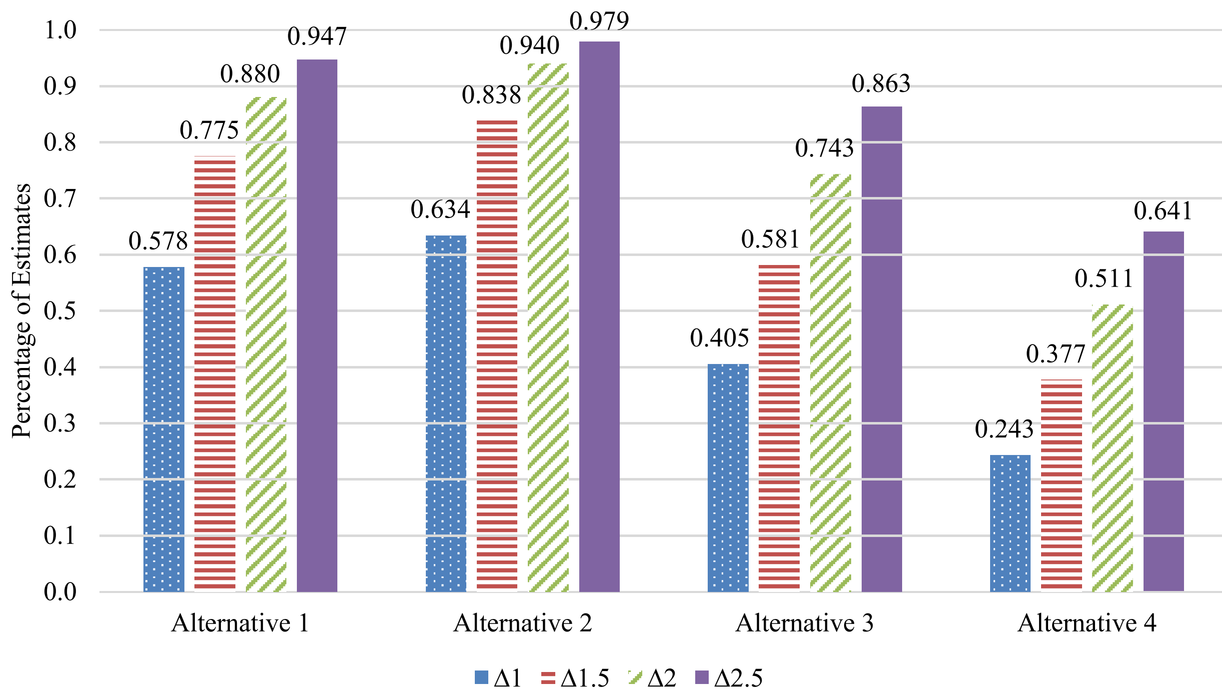

Figure 10. Chart. Comparison of alternative superelevation estimates.

Design Consistency Measures

A Policy on Geometric Design of Highways and Streets (the Green Book) includes "General Controls for Horizontal Alignment," some of which are related to achieving horizontal alignment design consistency:

- "Sharp curves should not be introduced at the ends of long tangents."
- "Where sharp curvature is introduced, it should be approached, where practical, by a series of successively sharper curves."

This study seeks to further inform users of this guidance. Two alignment indices that capture these design principles for achieving a consistent alignment are explored:

1. Ratio of tangent length to radius (RTR): the average length of the upstream and downstream tangents divided by the radius of the analysis curve. Estimated regression parameters corresponding to this variable are expected to be positive. Horizontal curves following longer tangents are expected to experience a larger number of roadway departure crashes than horizontal curves following shorter tangents. This effect is expected to be more pronounced for sharper curves, and less pronounced for flatter curves.
2. Modified change radius rate (CRR-M): the average radius of the upstream and downstream horizontal curves divided by the radius of the analysis curve. Estimated regression parameters corresponding to this variable are also expected to be positive. Horizontal curves following flatter curves are expected to experience a larger number of roadway departure crashes than horizontal curves following sharper curves. This effect is

expected to be more pronounced for sharper curves that follow flatter curves, and less pronounced for flatter curves that follow other flatter curves.

Summary

Table 8 provides a summary of variables available for modeling. The information in table 8 is applicable to both Washington and Utah datasets.

Table 8. Descriptive statistics for design consistency and roadway variables in Washington State and Utah.

Notation	Variable	Mean	S.D.	Minimum	Maximum
L	Segment length (mi)	0.169	0.109	0.02	1.143
AADT	Annual average daily traffic (veh/day) over the observation period	1,614	1,667	90.71	12,021
UTind	Indicator variable for segment location (= 1 if segment in Utah; 0 if segment in Washington)	0.628	0.484	0	1
Lc	Horizontal curve length (ft)	893.9	577.6	105.6	6,035
R	Horizontal curve radius (ft)	2,231	1,333	318.3	8,873
e	Superelevation (percent)	4.48	1.93	0.09	10
Defl	Deflection angle (degrees)	29.07	22.20	2.112	177.5
SpLmt	Speed limit (mph)	55.21	6.632	30	65
fd	Side friction demand (assuming speed = speed limit)	0.078	0.057	-0.002	0.407
UpTL	Length of upstream tangent (ft)	2,042	2,370	301.0	16,780
DnTL	Length of downstream tangent (ft)	2,079	2,305	300.5	15,835
AvUDTL	Average of upstream and downstream tangent length (ft)	2,060	1,929	308.7	15,333
UpR	Radius of upstream horizontal curve (ft)	2,247	1,362	250	9,950
DnR	Radius of downstream horizontal curve (ft)	2,292	1,514	318	11,460
AvUDR	Average of upstream and downstream curve radius (ft)	2,269	1,164	433	7,925
RTR	Ratio of tangent length to radius (ft/ft)	1.037	0.828	0.083	5.782
CRR-M	Modified change radius rate (ft/ft)	1.216	0.667	0.156	4.333

Crash Data

Crash data for Washington State were obtained from the HSIS for 2008-2012 (five years). Crash data for Utah were obtained from UDOT for 2008-2014 (seven years). Roadway departure crashes were identified as non-intersection crashes in which a vehicle crosses an edge line, centerline, or leaves the traveled way. In mapping this definition to variables on the Utah and Washington crash reports, roadway departure crashes were identified as those meeting both of the following criteria:

1. Non-intersection (i.e., not at-intersection or intersection-related); and
2. The first harmful event for at least one of the involved vehicles is a) ran-off-road (right or left), b) crossed centerline or median, c) went airborne, or d) hit fixed object.

The focus of the analysis was roadway departure crashes along horizontal curves on rural, two-lane highways. Each observation was represented by a single horizontal curve from the point of curvature (PC) to the point of tangency (PT). Roadway departure crashes occurring between the PC and PT of each curve were included in the analysis. The observed number of roadway departure crashes for each curve over the observation period were determined by severity:

- Fatal roadway departure crashes (RwD-K).
- Incapacitating injury roadway departure crashes (RwD-A).
- Non-incapacitating injury (RwD-B).
- Possible injury (RwD-C).
- No injury (RwD-O).

These counts were ultimately aggregated for modeling due to the low numbers into the following categories:

- Total (i.e., all severities) roadway departure crashes (RwD-KABCO).
- Fatal-plus-injury roadway departure crashes (RwD-KABC).

Table 9 and table 10 include summaries of descriptive statistics for the roadway departure crash counts associated with the horizontal curves included in this analysis.

Table 9. Descriptive statistics for roadway departure crash counts in Washington State (2008-2012).

Crash Type	N	Mean	S.D.	Min	Max	Rate (crashes/mi/year)	Rate (crashes/MVM)
RwD-KABCO	249	0.498	1.082	0	12	0.750	0.788
RwD_KABC	249	0.221	0.571	0	5	0.333	0.350

Table 10. Descriptive statistics for roadway departure crash counts in Utah (2008-2014).

Crash Type	N	Mean	Standard deviation	Min	Max	Rate (crashes/mi/year)	Rate (crashes/MVM)
RwD-KABCO	421	0.390	0.998	0	12	0.292	0.778
RwD_KABC	421	0.192	0.675	0	9	0.144	0.384

RESULTS AND DISCUSSION

Table 11, table 12, table 13, and table 14 summarize the model estimation results for the RwD-KABCO and RwD-KABC, respectively. Each table provides model estimation results for two different model specification alternatives. Variables capturing segment length, traffic volume, design consistency (modified change radius rate and ratio of tangent length to radius), and State (a State of Utah indicator variable) are included in both model specifications. The impact of horizontal curve radius is captured differently by the two models:

- Alternative 1 includes horizontal curvature effects through the specification of the inverse of horizontal curve radius (1/R).
- Alternative 2 includes horizontal curvature effects through the specification of side friction demand, which captures interactions between posted speed limit, horizontal curve radius, and superelevation.

Table 11. RwD-KABCO model estimation results for Alternative 1 (1/R).

Notation	Variable	Parameter estimate	S.E.
Ln(L*T)	Natural logarithm of segment length multiplied by time period	1.0 (offset)	n/a
Ln(AADT)	Natural logarithm of AADT	0.914**	0.103
1/R	Inverse of horizontal curve radius (ft^{-1})	860.36**	185.74
fd	Side friction demand (assuming speed = speed limit)	--	--
CRR-M	Modified change radius rate (ft/ft)	0.253**	0.118
RTR	Ratio of tangent length to radius (ft/ft)	0.037	0.096
UTind	Indicator variable for segment location (= 1 if segment in Utah; 0 if segment in Washington)	0.101	0.181
Constant	Model constant	-8.521**	0.848
alpha	Dispersion parameter	0.752**	0.192

Note: Alternative 1 (with 1/R): Likelihood ratio chi-squared, 5df = 154.67 ($p < 0.0001$); Pseudo R2 = 0.133; Likelihood ratio of alpha chi-squared, 1df = 44.08 ($p < 0.001$); ** statistically significant at 95 percent confidence level; * statistically significant at 85 percent confidence level; n/a not applicable; – not statistically significant.

Table 12. RwD-KABCO model estimation results for Alternative 2 (with fd).

Notation	Variable	Parameter estimate	S.E.
Ln(L*T)	Natural logarithm of segment length multiplied by time period	1.0 (offset)	n/a
Ln(AADT)	Natural logarithm of AADT	0.924**	0.105
1/R	Inverse of horizontal curve radius (ft^{-1})	--	--
fd	Side friction demand (assuming speed = speed limit)	4.292**	1.474
CRR-M	Modified change radius rate (ft/ft)	0.372**	0.119
RTR	Ratio of tangent length to radius (ft/ft)	0.049	0.101
UTind	Indicator variable for segment location (= 1 if segment in Utah; 0 if segment in Washington)	0.047	0.188
Constant	Model constant	-8.484**	0.869
alpha	Dispersion parameter	0.919**	0.208

Note: Alternative 2 (with fd): Likelihood ratio chi-squared, 5df = 142.76 ($p < 0.0001$); Pseudo R2 = 0.123; Likelihood ratio of alpha chi-squared, 1df = 61.00 ($p < 0.001$); ** statistically significant at 95 percent confidence level; * statistically significant at 85 percent confidence level; n/a not applicable; – not statistically significant.

Table 13. RwD-KABC model estimation results for Alternative 1 (with 1/R).

Notation	Variable	Parameter Estimate	S.E.
Ln(L*T)	Natural logarithm of segment length multiplied by time period	1.0 (offset)	n/a
Ln(AADT)	Natural logarithm of AADT	0.807**	0.142
1/R	Inverse of horizontal curve radius (ft^{-1})	801.18**	259.56
fd	Side friction demand (assuming speed = speed limit)	--	--
CRR-M	Modified change radius rate (ft/ft)	0.249*	0.169
RTR	Ratio of tangent length to radius (ft/ft)	0.176+	0.126
UTind	Indicator variable for segment location (= 1 if segment in Utah; 0 if segment in Washington)	0.072	0.257
Constant	Model constant	-8.579**	1.178
alpha	Dispersion parameter	1.299**	0.432

Note: Alternative 1 (with 1/R): Likelihood ratio chi-squared, 5df = 72.57 ($p < 0.0001$); Pseudo R2 = 0.1015; Likelihood ratio of alpha chi-squared, 1df = 27.62 ($p < 0.001$); ** statistically significant at 95 percent confidence level, * statistically significant at 85 percent confidence level, + statistically significant at 80 percent confidence level, n/a not applicable; – not statistically significant.

Table 14. RwD-KABC model estimation results for Alternative 2 (with fd).

Notation	Variable	Parameter Estimate	S.E.
Ln(L*T)	Natural logarithm of segment length multiplied by time period	1.0 (offset)	n/a
Ln(AADT)	Natural logarithm of AADT	0.816**	0.145
1/R	Inverse of horizontal curve radius (ft^{-1})	--	--
fd	Side friction demand (assuming speed = speed limit)	4.573**	2.047
CRR-M	Modified change radius rate (ft/ft)	0.369**	0.166
RTR	Ratio of tangent length to radius (ft/ft)	0.185+	0.133
UTind	Indicator variable for segment location (= 1 if segment in Utah; 0 if segment in Washington)	0.049	0.266
Constant	Model constant	-8.617**	1.208
alpha	Dispersion parameter	1.568**	0.464

Note: Alternative 2 (with fd): Likelihood ratio chi-squared, 5df = 68.49 ($p < 0.0001$); Pseudo R2 = 0.096; Likelihood ratio of alpha chi-squared, 1df = 36.86 ($p < 0.001$), ** statistically significant at 95 percent confidence level, * statistically significant at 85 percent confidence level, + statistically significant at 80 percent confidence level, n/a not applicable; – not statistically significant.

$$fd = \left(\frac{SpLmt^2}{15R} - 0.01e\right)$$

Figure 11. Equation. Side friction demand.

where:

fd = estimated side friction demand;

$SpLmt$ = speed limit (mph);

R = horizontal curve radius (ft); and

e = superelevation (percent).

Models that estimate horizontal curvature effects through the specification of the inverse of horizontal curve radius had slightly better fits, as measured by Pseudo R-squared, for both the RwD-KABCO and RwD-KABC models. Estimated regression parameters for both inverse radius and side friction demand variables were in the direction expected and statistically significant at the 95-percent confidence level for both the RwD-KABCO and RwD-KABC models. Alternative 2 includes horizontal curvature effects through the specification of side friction demand, which captures interactions between posted speed limit, horizontal curve radius, and superelevation, as shown in figure 11.

Design Consistency Effects

Estimated regression parameters for the modified change radius rate (CRR-M) were positive and statistically significant at either the 95-percent or 85-percent confidence level in all models. The results indicate that the expected number of roadway departure crashes increases as CRR-M increases. In other words, the expected number of roadway departure crashes on a horizontal curve not only increases as a function of that curve's radius, but also as a function of the radii on the upstream and downstream curves. The expected number of roadway departure crashes on the subject curve increases as the radius of that curve decreases. The sensitivity between the expected number of roadway departure crashes and radius is highest for smaller radii and the sensitivity continually decreases as the radii get larger and larger. Flatter upstream and downstream radii result in a higher number of roadway departure crashes on the subject curve, likely because drivers enter the subject curve after becoming used to the flatter radii and higher operating speeds of the surrounding curves. Sharper upstream and downstream radii result in a lower number of roadway departure crashes on the subject curve, likely because drivers enter the subject curve after becoming used to the sharper radii and lower operating speeds of the surrounding curves. The magnitudes of the CRR-M effects were similar for both the total and fatal-plus-injury crash models.

Estimated regression parameters for the RTR were positive, but were only statistically significant at the 85-percent confidence level in the fatal-plus-injury models. The results indicate that the expected number of fatal-plus-injury roadway departure crashes increases as RTR increases. In other words, the expected number of roadway departure crashes on a horizontal curve not only increases as a function of that curve's radius, but also as a function of the upstream and downstream tangent lengths. The expected number of roadway departure crashes on the subject curve increases as the radius of that curve decreases. The sensitivity between the expected number of roadway departure crashes and radius is highest for smaller radii and the sensitivity continually decreases as the radii get larger and larger. Longer upstream and downstream tangents result in a higher number of roadway departure crashes on the subject curve, likely because drivers enter the subject curve after becoming used to the longer tangent section of alignment and higher operating speeds. Shorter upstream and downstream tangents result in a lower number of roadway departure crashes on the subject curve, likely because drivers enter the subject curve without enough time to increase their speed or get used to driving on the tangent section of alignment. The fact that RTR is more related to fatal-plus-injury crashes than total crashes makes intuitive sense, as it is likely the design consistency measure most related to curve entering speeds.

Other Effects

The models demonstrate multiple other roadway departure safety effects in addition to those from the design consistency measures:

- Segment length and time period were included in the model as offset variables (i.e., setting those variables to unity provides an estimate of expected number of roadway departure crashes per mi per year).
- The models also include horizontal curve length as a covariate, and the estimated parameters indicate that the expected number of roadway departure crashes decrease as curve length increases. Curve length captures an interaction between deflection angle and curve radius, and the negative parameters seem to indicate that curve radius is the more important part of that interaction. Main effects of deflection angle were tested, but were very small and never statistically significant.
- The positive regression parameters associated with 1/R indicate that the expected number of roadway departure crashes on the subject curve increases as the radius of that curve decreases. The sensitivity between the expected number of roadway departure crashes and radius is highest for smaller radii and the sensitivity continually decreases as the radii get larger and larger. This result is expected and is statistically significant at the 95-percent confidence level for both the total and fatal-plus-injury crash models.
- The positive regression parameters associated with side friction demand indicate that the expected number of roadway departure crashes on the subject curve increases as the side friction demand increases. Side friction demand increases as the speed limit increases or as the radius of curve and/or superelevation decrease. This result is expected and is statistically significant at the 95-percent confidence level and 85-percent level for the total crash model and fatal-plus-injury crash model, respectively.
- Models included a State of Utah indicator variable to capture other unmeasured differences between Utah and Washington. The estimated parameter was always positive (indicating a higher number, on average, of reported roadway departure crashes in Utah) but never statistically significant.

A Closer Look at Estimated Radius Effects

This section provides additional discussion of model findings with respect to curve radius. The estimated coefficient for the inverse curve radius variable in the reported models leads to a radius CMF that is larger than the radius of curve CMF provided for rural, two-lane highways in the Highway Safety Manual (HSM).

Figure 12 illustrates a comparison of CMFs derived from the RwD-KABCO model reported in this chapter to the CMF in the HSM. Since the HSM provides a CMF for total crashes, the RwD-KABCO CMF was converted to a total crash CMF (shown as CMF MODEL). Figure 12 shows that the CMF derived from the RwD-KABCO model in this chapter is consistently larger than the HSM CMF. However, when considering a range defined by two-standard deviations from the model coefficient (shown as CMF MODEL 2 SD), the HSM CMF falls within the range of the confidence interval of the CMF derived in this effort. Further analysis revealed that the difference in study curve radii is likely responsible for much of the difference in the CMF magnitudes. The data used to develop the HSM CMF included curves with curve radii less than 1,000 ft, noted to be commonly as small as 200 ft. As noted in table 8, the average curve radius for this dataset was more than 2,200 ft. Overall, this comparison adds some confidence in the RwD-KABCO model results.

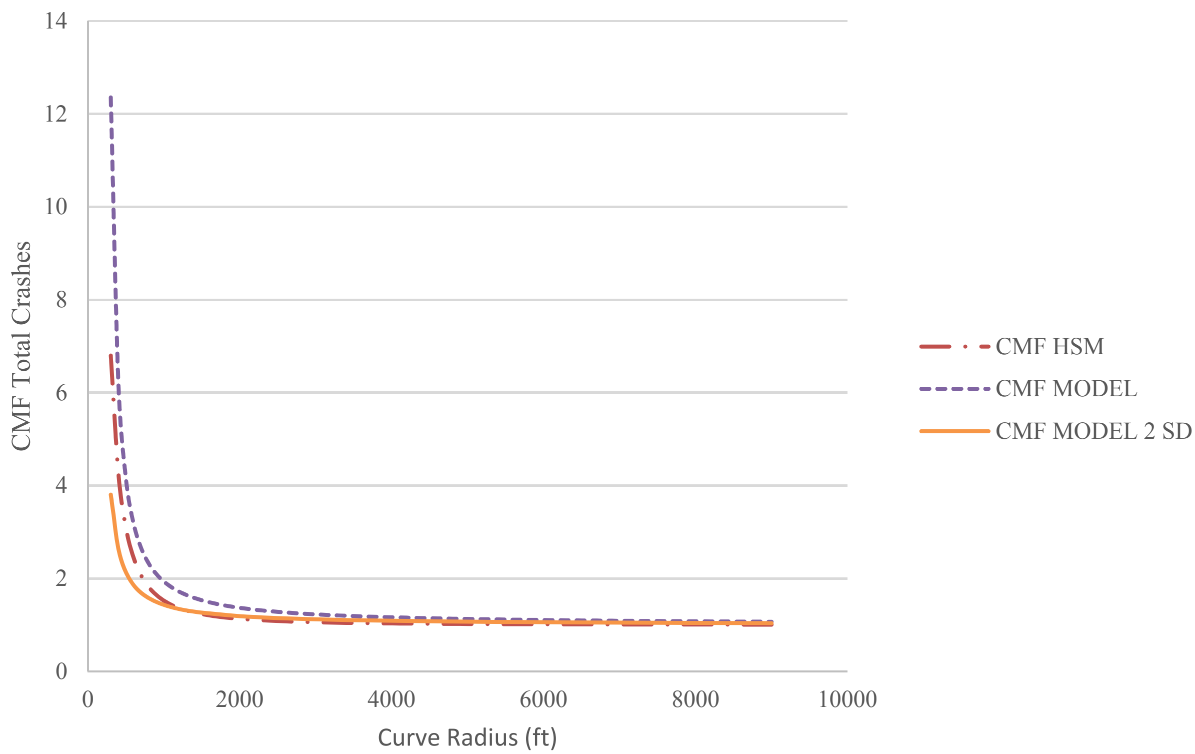

Figure 12. Graph. HSM and model derived CMFs for horizontal curve radius.

CONCLUSIONS AND RECOMMENDATIONS

This research explored and estimated relationships between alternative measures of design consistency and the expected number of roadway departure crashes along horizontal curves on rural, two-lane, two-way roads. The relationships between alternative measures of design consistency and the expected number of roadway departure crashes were explored using a NB regression modeling approach and observing the regression parameter estimates corresponding to the measures. Data used for model estimation were drawn from a concurrent FHWA "Predisposing Factors" project. (Porter et al. 2018) The data were adaptable to, but not originally collected for, studying design consistency. A few points regarding the methodology are worth noting in this context:

1. The study design was cross sectional. Whether estimated regression parameters from cross sectional studies can ever represent safety effects estimates is a matter of ongoing debate. More recent safety research has implemented cross sectional study designs intended to "mimic" randomized experiments and strengthen confidence in safety effects estimates, such as the propensity scores – potential outcomes framework. Road safety researchers have limited experience using the propensity scores – potential outcomes framework for treatments represented by continuous variables, such as alternative measures of design consistency. The adapted data and analysis scope did not support the use of the propensity scores – potential outcomes framework.
2. The sample of horizontal curves will impact the identified association between design consistency and expected number of roadway departure crashes. Since the modeling technique is non-linear, including larger radii curves will impact the coefficient estimates, which can lead to larger CMFs for small radii curves upon application.

The analysis results showed that the expected number of roadway departure crashes on a horizontal curve changes not only as a function of that curve's radius, but also as a function of the radii on the upstream and downstream curves and the upstream and downstream tangent lengths. More specifically, for a subject curve with a given set of characteristics:

- Flatter upstream and downstream radii result in a higher number of roadway departure crashes on the subject curve, likely because drivers enter the subject curve after becoming used to the flatter radii and higher operating speeds of the surrounding curves.
- Sharper upstream and downstream radii result in a lower number of roadway departure crashes on the subject curve, likely because drivers enter the subject curve after becoming used to the sharper radii and lower operating speeds of the surrounding curves.
- Longer upstream and downstream tangents result in a higher number of roadway departure crashes on the subject curve, likely because drivers enter the subject curve after becoming used to the longer tangent section of alignment and higher operating speeds.
- Shorter upstream and downstream tangents result in a lower number of roadway departure crashes on the subject curve, likely because drivers enter the subject curve without enough time to increase their speed or get used to driving on the tangent section of alignment.

Such findings are intuitive given the concept of design consistency and represents a potential advancement to existing predictive methods in the HSM, which estimates the expected number of crashes on a segment as a function of the characteristics of only that segment. Future work should attempt to replicate this analysis with a dataset specifically collected for that purpose.

CHAPTER 5. QUANTIFYING RELATIONSHIPS BETWEEN EXPECTED FREQUENCIES OF ROADWAY DEPARTURE CRASHES AND HORIZONTAL ALIGNMENT USING THE SHRP2 ROADWAY INFORMATION DATABASE

The objective of this study is to explore the transferability of previous findings (see Chapter 4 of this report) regarding the application of alignment indices as an indicator of the relationship between design consistency and safety. In keeping with past recommendations, the present study leveraged high-accuracy roadway data explicitly collected for research purposes. The following sections of this paper provide a brief overview of the data and methods applied to investigate the effect of design consistency on roadway departure crashes, as well as the conclusions and practical applications of design consistency for practitioners.

OVERVIEW OF DATA SOURCE

The project team obtained a copy of the Strategic Highway Research Program (SHRP) 2 RID 2.0 from the Center for Transportation Research and Education (CTRE) at Iowa State University. This dataset is derived from photogrammetry for assets (via windshield survey) and automated alignment processing using differential global positioning systems and inertial navigation. data collection effort. The SHRP 2 RID project team collected data in nine States along roads that were most frequently driven by participants in the previously conducted SHRP 2 Naturalistic Driving Study (NDS). These States include the following:

- Florida.
- Indiana.
- New York.
- North Carolina.
- Pennsylvania.
- Washington.

In addition to its compatibility with the NDS, the Roadway Information Database (RID) provides the opportunity to use its high-quality roadway inventory data for research on the effects of roadway design and traffic control strategies on highway safety. The RID includes a variety of high-resolution roadway attributes in geographic information systems (GIS) format. These data include the following:

- Horizontal curvature:
    - Radius.
    - Length.
    - Point of curvature (PC).
    - Point of tangency (PT).
    - Direction of curve (left or right based on driving direction).
- Vertical grade.
- Cross-slope/superelevation.
- Lanes: number, width, and type (through, turning, passing, acceleration, car pool, etc.).
- Shoulder type/curb (and paved width, if it exists).

- All Manual on Uniform Traffic Control Devices (MUTCD) signs.
- Guardrails/barriers.
- Intersections: location, number of approaches, and control (uncontrolled, all-way stop, two-way stop, yield, signalized, roundabout). Crossroad ramp termini are coded as intersections in the RID.
- Median presence: type (depressed, raised, flush, barrier).
- Rumble strip presence: location (center line, edge line, shoulder).
- Lighting presence.

These data are supplemented with other existing public datasets. Among them are the Highway Performance Monitoring System (HPMS), as well as State-level inventories of crash locations, average annual daily traffic (AADT), and incorporated areas.

DATA COLLECTION

The road segments used for analysis and modeling each consisted of one horizontal curve (i.e., each observation was defined by the observed number of roadway departure crashes occurring on an individual horizontal curve during the observation period). The analysis also considered characteristics of the "upstream" horizontal tangent, "upstream" horizontal curve, "downstream" horizontal tangent, and "downstream" horizontal curve.

This section outlines the method used for collecting data associated with eligible horizontal curves and their associated roadway tangents in Pennsylvania and Indiana. The study team identified these two States as the most likely to have rural, two-lane horizontal curves within the RID dataset. Eligible study curves met the following criteria:

- Unincorporated, rural location.
- Two, undivided through lanes only (no dedicated turn or acceleration lanes).
- Bidirectional.
- Subject horizontal curve is not within 300 ft of another horizontal curve (i.e., each upstream and downstream tangent approaching a horizontal curve is at least 300 ft in eligible tangent roadway length: two through lanes with no dedicated turn lanes).

Horizontal Curve Collection

The SHRP 2 RID 2.0 stores horizontal curves in a bidirectional format, with two linear elements representing each travel direction along the curve. Attributes associated with each curve direction are stored within the existing horizontal curve feature class for each State. The project team queried the Pennsylvania and Indiana RID geodatabases to identify eligible rural, two-lane curves using the python-based Dynamic Segmentation Tool developed by CTRE. The team then compared the reduced curve output dataset against the entire State RID inventory to remove rural, two-lane curves that fell within 300 ft of an adjacent horizontal curve (i.e., one or both of the approach tangents were less than 300 ft in length).

The project team assigned a unique ID to each directional pair of eligible curves. This allowed the team to merge directional attributes into a single curve observation. The project team utilized the unique curve ID to generate average values along the curve and reconcile attribute differences between travel directions. The team determined the following attributes for each merged curve observation:

- Average radius.
- Average grade.
- Average curve length.
- Presence of a superelevated roadway or normal crown.
- Indicators for presence of roadway elements.
    - Speed limit.
    - Advisory speed.
    - Curve warning signs.
    - Curve chevrons.
    - Rumble strips.
    - Roadside barriers.
    - Intersections.
    - Lighting.

Each study curve was associated with an upstream and downstream curve. The order was determined by geographic orientation, with upstream curves being south or west of the observation curve and downstream curves being north or east of the study curve. The team linked the average radius, average grade, and average curve length of the upstream/downstream curves with the study curve. Figure 9 illustrates the orientation of study curves with their upstream and downstream components.

Tangent Collection

The project team defined tangents as the entire stretch of roadway between a study curve and its upstream and downstream counterparts. The team did not apply any eligibility criteria to identifying tangents besides that which was necessary to determine eligible curves. The team only collected the length, the average cross-slope, and the average grade of the tangent. These data were associated and oriented with the study curves in the same way as the upstream/downstream curves.

Crash Data

Pennsylvania and Indiana State crash data between the years 2006 and 2013 (inclusive) were provided as part of the RID. This analysis included crashes between 2008 and 2013 (inclusive). The project team defined curve-related crashes as those that occurred within 150 ft upstream of the PC or PT or those that fell within the limits of the horizontal curve. Applying the Federal Highway Administration's (FHWA) definition of a roadway departure crash, the team identified roadway departure crashes using the manner of collision variable associated with each State's crash data. The collision codes included in the roadway departure analysis are provided in table 15.

Table 15. Collision codes for roadway departure crashes.

Pennsylvania	Indiana
0: Non-Collision	2: Head-On
2: Head-On	4: Sideswipe (same direction)
5: Sideswipe (same direction)	5: Sideswipe (opposite direction)
6: Sideswipe (opposite direction)	6: Ran off road
7: Hit fixed object	13: Non-Collision

These crash types were sorted into the following two categories for further analysis: (1) the total number of roadway departure crashes and (2) the number of fatal and injury roadway departure crashes along a given curve. While Pennsylvania crash data included the total (i.e., all severities) roadway departure crashes (KABCO) designation for maximum injury severity, the available Indiana dataset only referenced fatal, injury, and property damage only (PDO) crashes. The project team combined all fatal and injury crashes to allow for comparison between States. Table 16 and table 17 summarize roadway departure and fatal and injury roadway departure crashes by State.

Table 16. Roadway departure crash summary statistics for Indiana (2008-2013).

Variable	Notation	N	Mean	S.D.	Minimum	Maximum
RD Crashes	RDWDP_total	466	2.02	3.14	0	28
FI RD Crashes	RDWDP_KABC	466	0.59	1.11	0	8

The roadway departure and fatal and injury roadway departure crash rates for Pennsylvania were 1.09 crashes/mi/year and 0.580 crashes/mi/year, respectively. The roadway departure and fatal and injury roadway departure crash rates for Indiana were 1.67 crashes/mi/year and 0.49 crashes/mi/year, respectively.

Table 17. Roadway departure crash summary statistics for Pennsylvania (2008-2013).

Variable	Notation	N	Mean	S.D.	Minimum	Maximum
RD crashes	RDWDP_total	423	1.15	1.34	0	7
FI RD crashes	RDWDP_KABC	423	0.61	0.88	0	5

The roadway departure and fatal and injury roadway departure crash rates for Pennsylvania were 1.09 crashes/mi/year and 0.580 crashes/mi/year, respectively. The roadway departure and fatal and injury roadway departure crash rates for Indiana were 1.67 crashes/mi/year and 0.49 crashes/mi/year, respectively.

AADT Collection

Pennsylvania AADT data were available between 2011 and 2013 (inclusive). Indiana AADT data were available between 2009 and 2012 (inclusive). The project team assumed that the earliest or latest observed AADT values were valid proxies for any study period years outside of the range for which data were available. This approach is consistent with that in the Part C "Predictive Method" chapters of the *Highway Safety Manual* (HSM). (AASHTO 2010)

Summary

Table 18 provides the summary statistics for data associated with the study curves in Indiana. Table 19 provides the summary statistics for data associated with the study curves in Pennsylvania. For both States, the sample size is smaller for the upstream and downstream tangents and curves; this is because in some cases, data were only available for one direction (upstream or downstream). In those cases, the study curve was dropped from the analysis.

As with the Utah and Washington analysis, this effort explores two alignment indices that capture principles for achieving a consistent alignment:

1. Ratio of tangent length to radius (RTR): the average length of the upstream and downstream tangents divided by the radius of the analysis curve. Estimated regression parameters corresponding to this variable are expected to be positive. Horizontal curves following longer tangents are expected to experience a larger number of roadway departure crashes than horizontal curves following shorter tangents. This effect is expected to be more pronounced for sharper curves, and less pronounced for flatter curves.
2. Modified change radius rate (CRR-M): the average radius of the upstream and downstream horizontal curves divided by the radius of the analysis curve. Estimated regression parameters corresponding to this variable are also expected to be positive. Horizontal curves following flatter curves are expected to experience a larger number of roadway departure crashes than horizontal curves following sharper curves. This effect is expected to be more pronounced for sharper curves that follow flatter curves, and less pronounced for flatter curves that follow other flatter curves.

Table 18. Indiana horizontal curve summary statistics.

Variable	Notation	N	Mean	S.D.	Minimum	Maximum
Curve radius (ft)	R	466	2,581.8	2,121.5	97.5	16,769.5
Curve grade (degrees)	G	466	1.4	1.4	0.0	7.9
Curve length (ft)	L	466	1,055.1	566.3	129.0	4,059.5
Curve superelevation (degrees)	e	466	3.6	1.8	0.3	9.6
AADT (veh/day)	AADT	466	4,595.4	2,845.2	224.4	27,224.2
Total curve lane width (ft)	Lane_Wid	466	20.0	2.7	11.8	25.7
Total curve shoulder width (ft)	Should_Wid	466	5.9	5.9	0	25.6
Presence of an intersection	Intersect	466	0.4	0.5	0	1
Normal crown	Norm_Cn	466	0.2	0.4	0	1
Deflection angle (degrees)	Defl	466	32.0	18.2	5.3	108.1
Side-friction demand	Fd	466	0.1	0.2	0.0	1.9
Speed limit (mph)	SpLmt	466	51.0	6.1	20.0	55.0
Length of upstream tangent (ft)	UpTL	456	2,635.2	6,664.0	312.0	117,674.7
Length of downstream tangent (ft)	DnTL	456	2,444.6	4,238.0	310.1	39,423.4
Average length of upstream/ downstream tangent (ft)	AvUDTL	456	2,539.9	4,051.4	351.1	62,327.8
Radius of upstream horizontal curve (ft)	UpR	456	2,728.6	2,429.7	62.0	16,894.5
Radius of downstream horizontal curve (ft)	DnR	456	2,494.3	2,178.2	42.0	16,894.5
Average radius of upstream/downstream radius (ft)	AvUDR	456	2,611.4	1,791.6	174.0	10,305.8
Ratio of tangent length to radius (ft/ft)	RTR	456	1.7	5.0	0.1	74.3
Modified change radius rate (ft/ft)	CRR-M	456	1.5	1.9	0.03	29.6

Table 19. Pennsylvania horizontal curve summary statistics.

Variable	Notation	N	Mean	S.D.	Minimum	Maximum
Average radius	R	423	3,423.7	3,155.1	97.5	16,221.0
Average grade	G	423	2.0	1.8	0.0	12.4
Average curve length	L	423	915.3	508.5	141.0	4,457.0
Curve superelevation (degrees)	e	423	3.7	1.9	0.9	10.8
AADT (veh/day)	AADT	423	3,847.2	2,363.8	15.0	12,904.8
Total curve lane width (ft)	Lane_Wid	423	20.9	1.3	17.1	27.0
Total curve shoulder width (ft)	Should_Wid	423	6.2	3.5	0	16.9
Presence of an intersection	Intersect	423	0.6	0.5	0	1
Normal crown	Norm_Cn	423	0.3	0.4	0	1
Deflection angle (degrees)	Defl	423	24.8	18.9	5.4	124.0
Side-friction demand	Fd	423	0.1	0.1	0.0	0.8
Speed limit (mph)	SpLmt	423	47.7	8.0	25.0	55.0
Length of upstream tangent (ft)	UpTL	398	1,811.9	1,823.4	312.1	14,579.4
Length of downstream tangent (ft)	DnTL	398	1,917.7	2,152.2	304.6	15,378.3
Average of length upstream/ downstream tangent (ft)	AvUDTL	398	1,864.8	1,524.4	319.6	11,462.6
Radius of upstream horizontal curve (ft)	UpR	398	3,218.6	3,187.6	35.0	22,150.5
Radius of downstream horizontal curve (ft)	DnR	398	3,062.2	2,820.3	78.0	15,889.0
Average radius of upstream/downstream radius (ft)	AvUDR	398	3,140.4	2,307.0	333.5	11,675.8
Ratio of tangent length to radius (ft/ft)	RTR	398	0.9	0.9	0.05	8.2
Modified change radius rate (ft/ft)	CRR-M	398	1.4	1.7	0.13	25.9

DATA ASSESSMENT FOR MODELING

The information available to the project team for defining roadway departure crashes differed between the States, particularly related to the States' handling of non-collisions. Pennsylvania non-collision crashes could be further differentiated to identify specific types of non-collision crashes, while the Indiana non-collision crashes could not. Non-collision crashes typically include overturn/rollover crashes, struck by falling object crashes, and other non-collision crashes. Drawing on wording in the crash coding manuals provided with the RID, other non-collision crashes may include sudden stops causing an occupant to be injured or breakage of a vehicle part resulting in property damage or injury.

In Pennsylvania, non-collision crashes accounted for approximately seven percent of total crashes. In Indiana, non-collision crashes accounted for approximately two percent of total crashes. In Pennsylvania, overturn-rollover crashes accounted for 65 percent of non-collision crashes. In Indiana, the study team could not further differentiate types of non-collision crashes. The project team felt that it was important to include overturn/rollover crashes in the dataset, but could not separate those crashes from other non-collision crashes that were not related to roadway departures in Indiana. Therefore, the project team developed one set of roadway departure crashes that included non-collision crashes and one set of roadway departure crashes that excluded non-collision crashes to test in the models.

Because non-collision crashes accounted for a small percentage of overall crashes, the models indicated no meaningful differences between the datasets that included or excluded them. Therefore, to slightly bolster sample size, and to include all roadway departure crashes, the project team opted to include non-collision crashes in the analysis dataset.

Additionally, the project team examined the data from Pennsylvania and Indiana to determine if the two States' data could be pooled together for analysis. Table 20 and table 21 provide a model for design consistency with coefficients estimated separately for Pennsylvania and for Indiana. The coefficients for all variables except for the presence of an intersection and the ratio of tangent length to radius are similar. The presence of an intersection has no effect in roadway departure crashes in Pennsylvania and the ratio of average tangent length to radius has no apparent effect in Indiana. With the relative consistency between States, the project team determined that the States could be combined for further analysis, with special attention paid to pooled model parameters for intersection presence and the ratio of average tangent length to radius.

Table 20. Design consistency analysis for Pennsylvania.

Measure	Coef.	L95	U95
Log-AADT	0.354	0.138	0.571
Intersection presence	-0.017	-0.250	0.216
Average vertical grade	0.067	0.004	0.130
Total lane width	-0.038	-0.144	0.068
Total shoulder width	-0.024	-0.065	0.017
CRR_M	0.098	0.018	0.178
RTR	0.336	0.205	0.466
Constant	-2.357	-4.629	-0.085

Table 21. Design consistency analysis for Indiana.

Measure	Coef.	L95	U95
Log-AADT	0.383	0.160	0.606
Intersection presence	0.824	0.570	1.078
Average vertical grade	0.063	-0.035	0.160
Total lane width	-0.055	-0.103	-0.008
Total shoulder width	-0.052	-0.075	-0.028
CRR_M	0.107	-0.001	0.215
RTR	0.007	-0.037	0.051

Radius Functional Form

The project team explored the functional form of horizontal curve radius for inclusion in the models. The team used Variable Introduction Exploratory Data Analysis (VIEDA) as described by Hauer (2014) in his book *The Art of Regression Modeling in Road Safety*. The project team developed a prediction model for roadway departure crashes containing predictors other than horizontal curve radius. The base model included an offset variable (accounting for curve length and number of data years), AADT, presence of an intersection, average vertical grade, total lane width, total shoulder width, and an indicator for Pennsylvania. The project team saved the predicted number of crashes for each curve over the period and compared the predicted crashes to observed crashes, aggregating curves into bins by radius in 500-ft increments.

Figure 13 provides a plot of the ratio of observed to predicted crashes for each 500-ft bin from 250 ft to 8,250 ft (as measured by the center of each bin). A value greater than 1.0 indicates that the model under-predicts crashes for a set of curves and a value less than 1.0 indicates that the model over-predicts crashes for a set of curves. In general, the base model appears to under-predict for smaller radius curves and over-predicts for flatter curves (indicating the horizontal curve radius does indeed have an impact on safety performance). The shape generally resembles a power function.

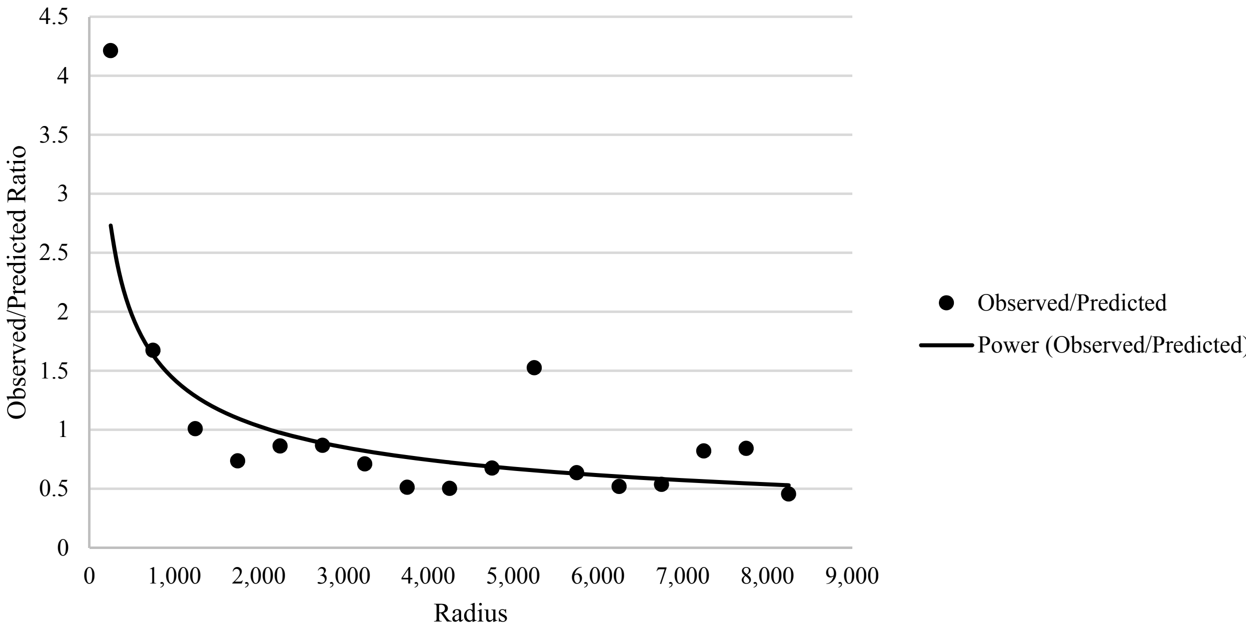

Figure 13. Graph. Observed to predicted crash ratio by radius.

The project team used this information to compare several functional forms for horizontal curve radius, including exponential, power, and polynomial forms. The team also explored the inverse of the horizontal curve radius, since the inverse radius flattens toward zero as radius nears infinity (allowing for a smooth transition to tangent segments). The inverse radius form allows for greater flexibility and range for the impact of the "elbow" in crashes that generally occurs on horizontal curves (i.e., once a certain radius threshold is reached, crash frequency increases at a faster rate, but the relationship is generally flat for radii above this threshold). Figure 14 compares crash modification factors (CMFs) developed from models including each functional form of horizontal curve radius. The exponential form of horizontal curve radius was found to have the worst fit to the data and the least flexibility as a CMF. The power function fit the data approximately the same as the inverse curve radius, but the CMF elbow is less flexible. This function also includes a long, broad range of CMFs greater than 2.0. A fifth-degree polynomial was found to fit the data the best, and provided a generally reasonable CMF across the range of horizontal curve radii; however, this functional form provides little practical value, is difficult to interpret, and includes small fluctuations in CMFs for larger values of horizontal curve radius. Therefore, the project team chose to proceed with the inverse horizontal curve radius specification for final models.

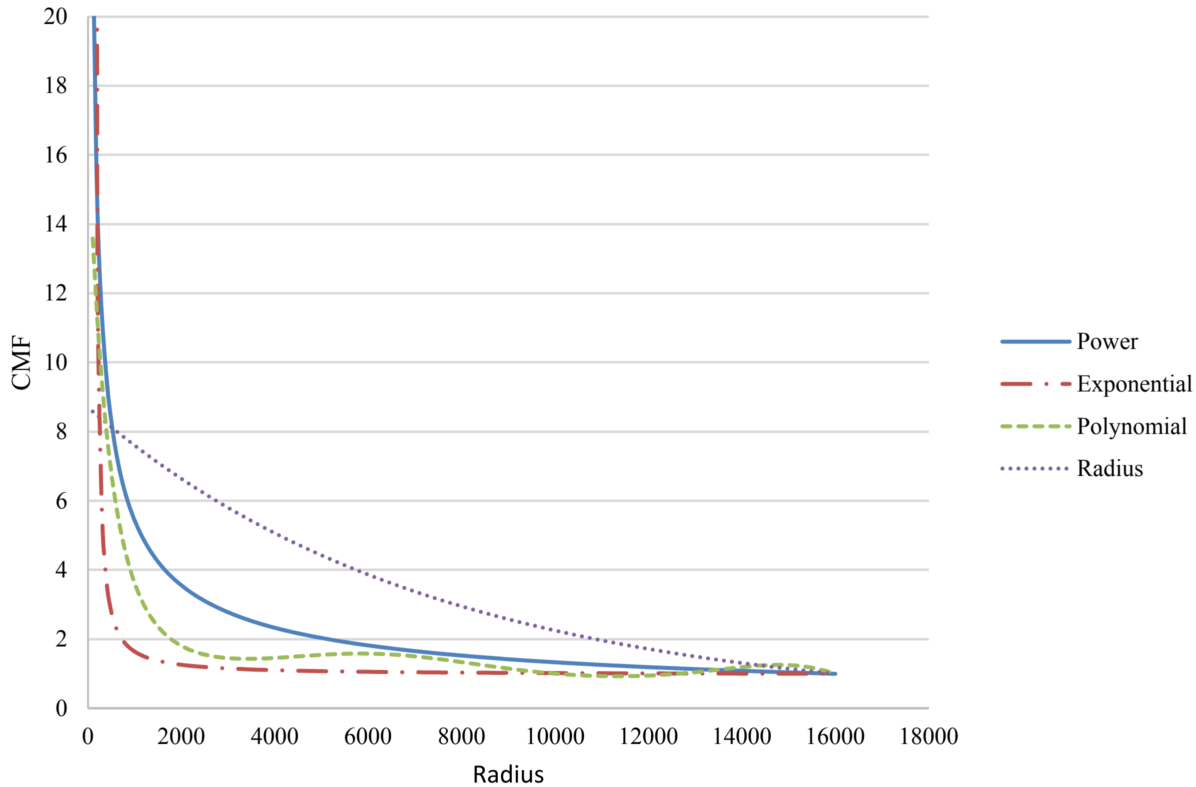

Figure 14. Graph. Comparison of CMFs by functional form.

RESULTS AND DISCUSSION

The project team was interested in estimating the safety effects of horizontal curve radius, superelevation rate, and design consistency on roadway departure crashes. However, as table 22 shows, there is substantial correlation among variables of interest. The radius of the study curve is included in the calculation of the modified change radius rate, ratio of average tangent length to radius, and friction demand. Therefore, it is unsurprising that inverse radius is more than 50 percent correlated to each of these other characteristics. Due to this complication, the project team chose to estimate the safety impacts of these features separately.

Table 22. Correlation among predictors of interest.

--	Inverse Radius	CRR_M	RTR	fd
Inverse Radius	1.000	0.590	0.550	0.880
CRR_M	0.590	1.000	0.422	0.512
RTR	0.550	0.422	1.000	0.657
fd	0.880	0.512	0.657	1.000

Table 23,

table 24, table 25, table 26, table 27, and table 28 provide separate models based on the inclusion of horizontal curve radius and superelevation, friction demand, and design consistency, respectively. Each of the tables include two models, one for roadway departure crashes of all severities and one for fatal and injury roadway departure crashes. Each of the models includes variables for curve length, AADT, presence of an intersection, total lane width, total shoulder width, and an indicator for unobserved differences between Pennsylvania and Indiana. Figure 11 provides an overview of the calculation of friction demand.

Table 23. RwD-KABCO for inverse radius and normal crown.

Notation	Variable	Parameter estimate	S.E.
LN(L*T)	Natural logarithm of segment length multiplied by time period	1.0 (offset)	n/a
Ln(AADT)	Natural logarithm of AADT	0.598**	0.078
1/R	Inverse of horizontal curve radius (ft^{-1})	464.66**	55.77
Norm_Cn	Indicator for normal crown (=1 if normal crown; 0 if superelevated)	-0.309**	0.135
Norm_Cn* 1/R	Interaction between indicator for normal crown and inverse radius	610.78**	171.08
Intersect	Indicator for intersection presence (=1 if present; 0 if not present)	0.486**	0.087
Grade	Absolute value of average grade on horizontal curve (percent)	0.039*	0.028
Lane_Wid	Total width of combined lanes on horizontal curve (ft)	-0.031*	0.020
Should_Wid	Total width of combined shoulders on horizontal curve (ft)	-0.034**	0.010
PAind	Indicator for segment location (=1 if segment in Pennsylvania; 0 if segment in Indiana)	-0.234**	0.094
Constant	Model constant	-4.275**	0.722
Alpha	Dispersion parameter	0.805**	0.077

Note: Alternative 1 (RwD-KABCO): Likelihood ratio chi-squared, 9df = 236.23 ($p < 0.0001$); Pseudo R2 = 0.075; Likelihood ratio of alpha chi-squared, 1df = 458.02 ($p < 0.001$; Pseudo R2 = 0.069; Likelihood ratio of alpha chi-squared, 1df = 65.85 ($p < 0.001$); ** statistically significant at 95 percent confidence level; * statistically significant at 80 percent confidence level; n/a not applicable.

Table 24. RwD-KABC for inverse radius and normal crown.

Notation	Variable	Parameter Estimate	S.E.
LN(L*T)	Natural logarithm of segment length multiplied by time period	1.0 (offset)	n/a
Ln(AADT)	Natural logarithm of AADT	0.580**	0.099
1/R	Inverse of horizontal curve radius (ft^{-1})	405.32**	58.99
Norm_Cn	Indicator for normal crown (=1 if normal crown; 0 if superelevated)	-0.496**	0.167
Norm_Cn* 1/R	Interaction between indicator for normal crown and inverse radius	512.07**	185.30
Intersect	Indicator for intersection presence (=1 if present; 0 if not present)	0.310**	0.110
Grade	Absolute value of average grade on horizontal curve (percent)	0.054*	0.033
Lane_Wid	Total width of combined lanes on horizontal curve (ft)	-0.037*	0.027
Should_Wid	Total width of combined shoulders on horizontal curve (ft)	-0.058**	0.014
PAind	Indicator for segment location (=1 if segment in Pennsylvania; 0 if segment in Indiana)	0.416**	0.121
Constant	Model constant	-4.960**	0.930
Alpha	Dispersion parameter	0.705**	0.129

Note: Pseudo R2 = 0.075; Likelihood ratio of alpha chi-squared, 1df = 458.02 ($p < 0.001$); Alternative 2 (RK-KABC): Likelihood ratio chi-squared, 9df = 134.42 ($p < 0.0001$); Pseudo R2 = 0.069; Likelihood ratio of alpha chi-squared, 1df = 65.85 ($p < 0.001$); ** statistically significant at 95 percent confidence level; * statistically significant at 80 percent confidence level; n/a not applicable.

Table 25. RwD-KABCO for friction demand.

Notation	Variable	Parameter estimate	S.E.
LN(L*T)	Natural logarithm of segment length multiplied by time period	1.0 (offset)	n/a
Ln(AADT)	Natural logarithm of AADT	0.553**	0.079
fd	Side friction demand (assuming speed = speed limit)	4.101**	0.464
Intersect	Indicator for intersection presence (=1 if present; 0 if not present)	0.508**	0.088
Grade	Absolute value of average grade on horizontal curve (percent)	0.064**	0.028
Lane_Wid	Total width of combined lanes on horizontal curve (ft)	-0.047**	0.020
Should_Wid	Total width of combined shoulders on horizontal curve (ft)	-0.030**	0.010
PAind	Indicator for segment location (=1 if segment in Pennsylvania; 0 if segment in Indiana)	-0.220**	0.096
Constant	Model constant	-3.664**	0.723
Alpha	Dispersion parameter	0.859**	0.080

Note: Alternative 1 (RwD-KABCO): Likelihood ratio chi-squared, 7df = 208.24 ($p < 0.0001$); Pseudo R2 = 0.066; Likelihood ratio of alpha chi-squared, 1df = 503.21 ($p < 0.001$);; Pseudo R2 = 0.056; Likelihood ratio of alpha chi-squared, 1df = 80.79 ($p < 0.001$); ** statistically significant at 95 percent confidence level; * statistically significant at 80 percent confidence level; n/a not applicable.

Table 26. RwD-KABC for friction demand.

Notation	Variable	Parameter Estimate	S.E.
LN(L*T)	Natural logarithm of segment length multiplied by time period	1.0 (offset)	n/a
Ln(AADT)	Natural logarithm of AADT	0.540**	0.101
fd	Side friction demand (assuming speed = speed limit)	3.346**	0.496
Intersect	Indicator for intersection presence (=1 if present; 0 if not present)	0.329**	0.113
Grade	Absolute value of average grade on horizontal curve (percent)	0.083**	0.033
Lane_Wid	Total width of combined lanes on horizontal curve (ft)	-0.049*	0.027
Should_Wid	Total width of combined shoulders on horizontal curve (ft)	-0.052**	0.014
PAind	Indicator for segment location (=1 if segment in Pennsylvania; 0 if segment in Indiana)	0.395**	0.123
Constant	Model constant	-4.504**	0.936
Alpha	Dispersion parameter	0.801**	0.136

Note: Pseudo R2 = 0.066; Likelihood ratio of alpha chi-squared, 1df = 503.21 ($p < 0.001$); Alternative 2 (RK-KABC): Likelihood ratio chi-squared, 7df = 108.65 ($p < 0.0001$); Pseudo R2 = 0.056; Likelihood ratio of alpha chi-squared, 1df = 80.79 ($p < 0.001$); ** statistically significant at 95 percent confidence level; * statistically significant at 80 percent confidence level; n/a not applicable.

Table 27. RD models for design consistency.

Notation	Variable	Parameter Estimate	S.E.
LN(L*T)	Natural logarithm of segment length multiplied by time period	1.0 (offset)	n/a
Ln(AADT)	Natural logarithm of AADT	0.429**	0.082
CRR-M	Modified change radius rate (ft/ft)	0.165**	0.039
RTR	Ratio of tangent length to radius (ft/ft)	0.018	0.024
Intersect	Indicator for intersection presence (=1 if present; 0 if not present)	0.465**	0.093
Grade	Absolute value of average grade on horizontal curve (percent)	0.076**	0.030
Lane_Wid	Total width of combined lanes on horizontal curve (ft)	-0.056**	0.021
Should_Wid	Total width of combined shoulders on horizontal curve (ft)	-0.043**	0.010
PAind	Indicator for segment location (=1 if segment in Pennsylvania; 0 if segment in Indiana)	-0.308**	0.101
Constant	Model constant	-2.224**	0.728
Alpha	Dispersion parameter	0.965**	0.088

Note: Alternative 1 (RwD-KABCO): Likelihood ratio chi-squared, 8df = 144.06 ($p < 0.0001$); Pseudo R2 = 0.047; Likelihood ratio of alpha chi-squared, 1df = 524.34 ($p < 0.001$); Likelihood ratio of alpha chi-squared, 1df = 82.25 ($p < 0.001$); ** statistically significant at 95 percent confidence level; * statistically significant at 80 percent confidence level; n/a not applicable.

Table 28. RwD-KABC for design consistency.

Notation	Variable	Parameter Estimate	S.E.
LN(L*T)	Natural logarithm of segment length multiplied by time period	1.0 (offset)	n/a
Ln(AADT)	Natural logarithm of AADT	0.456**	0.104
CRR-M	Modified change radius rate (ft/ft)	0.136**	0.042
RTR	Ratio of tangent length to radius (ft/ft)	0.008	0.018
Intersect	Indicator for intersection presence (=1 if present; 0 if not present)	0.272**	0.116
Grade	Absolute value of average grade on horizontal curve (percent)	0.091**	0.034
Lane_Wid	Total width of combined lanes on horizontal curve (ft)	-0.054*	0.028
Should_Wid	Total width of combined shoulders on horizontal curve (ft)	-0.063**	0.014
PAind	Indicator for segment location (=1 if segment in Pennsylvania; 0 if segment in Indiana)	0.338**	0.126
Constant	Model constant	-3.533	0.941
Alpha	Dispersion parameter	0.875**	0.149

Note: Alternative 2 (RK-KABC): Likelihood ratio chi-squared, 8df = 74.69 ($p < 0.0001$); Pseudo R2 = 0.040; Likelihood ratio of alpha chi-squared, 1df = 82.25 ($p < 0.001$); ** statistically significant at 95 percent confidence level; * statistically significant at 80 percent confidence level; n/a not applicable.

Radius and Normal Crown Effects

The estimated regression parameters for the safety effect of inverse horizontal curve radius were significant at the 95-percent confidence level and indicated that expected number of roadway departure and fatal and injury roadway departure crashes increases as horizontal curve radius decreases. The sensitivity between the expected number of roadway departure crashes and radius is highest for smaller radii and the sensitivity continually decreases as the radii get larger and larger. The main effects for the presence of a normal crown and for the interaction between a normal crown and inverse horizontal curve radius are significant at the 95-percent confidence level. While the main effect is significant, it has no bearing on the CMF for the change in radius from an existing condition to a proposed alternative condition if a normal crown is carried through the curve in both conditions. However, it is used in the CMF if the proposed condition and existing result in a change from a normal crown to a superelevated cross section. The CMF for horizontal curvature, with or without a normal crown, is calculated using the following equation in figure 15.

$$CMF = \frac{exp^{\left(\frac{464.66}{R_{proposed}} - 0.309 \times NC_{proposed} + \frac{610.78 \times NC_{proposed}}{R_{proposed}}\right)}}{exp^{\left(\frac{464.66}{R_{existing}} - 0.309 \times NC_{existing} + \frac{610.78 \times NC_{exis\ ting}}{R_{existing}}\right)}}$$

Figure 15. Equation. CMF for horizontal curvature.

Figure 16 provides a plot of the CMF by curve radius for a normal crown and a superelevated condition for RwD-KABCO. The plots show that the CMF is more responsive to radius on a normal crown roadway, which is expected. Additionally, the plot compares the CMF to the horizontal curve radius CMF from the HSM. The HSM radius CMF used the average curve length from the curves in this study as an input. The CMF developed from this dataset is larger than the CMF from the HSM. The CMFs are not directly comparable, as the HSM CMF is for all crash types.

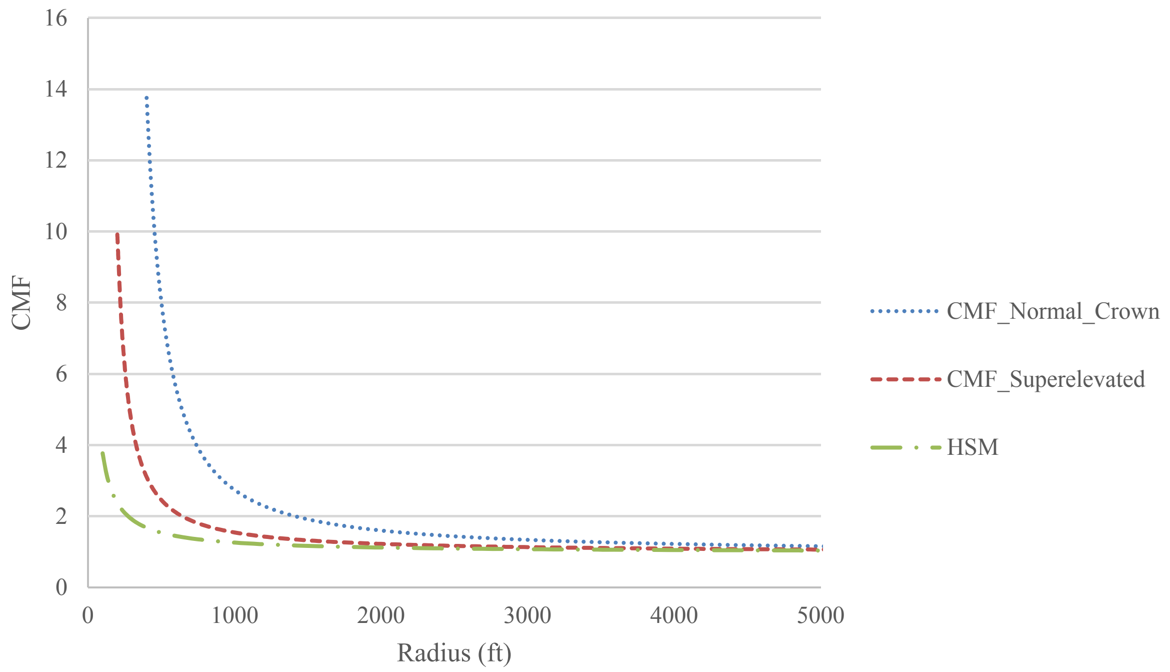

Figure 16. Graph. CMF for changing radius (normal crown versus superelevated versus HSM CMF).

The project team considered the coefficient for inverse horizontal curve radius for all curves, curves with a radius of 1,500 ft or smaller, and curves with a radius of 1,000 ft or smaller. Figure 17 shows the CMFs for horizontal curve radius, assuming a base condition of a 1,000-ft radius, using the results of each model. The results show that when the average curve included in the analysis is larger, the estimated effect of horizontal curvature is larger. Therefore, the average effect of curvature is dependent upon the average of the sample being used. For the RID analysis, the average curve had a radius of over 2,000 ft, so it is possible that the analysis in table 23 and table 24 overestimates the safety effects of horizontal curve radius for tighter horizontal curves. Since smaller horizontal curve radii are typically of interest in safety analyses, a large sample of smaller radius curves should be explored in future analysis datasets. The overestimation challenge also appeared to be addressed by interacting curve radius with speed and superelevation as described in the next section.

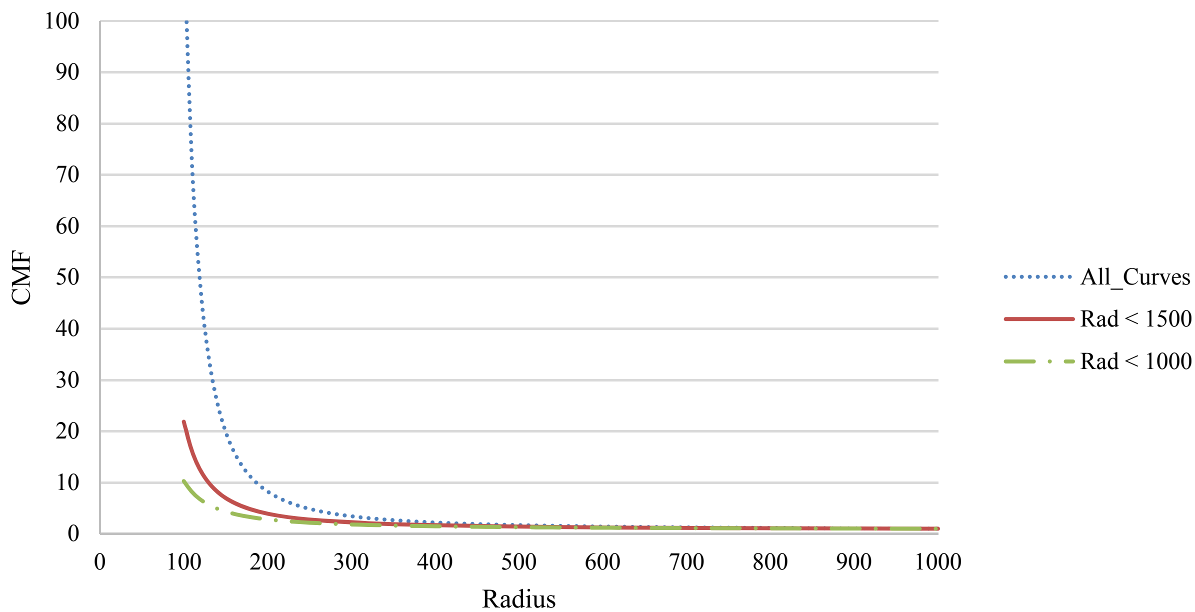

Figure 17. Graph. Comparison of CMFs by curves included in analysis.

Friction Demand Effects

The estimated regression parameter for friction demand is significant at the 95-percent confidence level. The results indicate that the expected number of roadway departure and fatal and injury roadway departure crashes increases as friction demand increases. Friction demand, as shown in figure 11, is calculated through the point-mass model, with speed limit substituting for design speed. Under this assumption, the expected number of roadway departure and fatal and injury roadway departure crashes increases as posted speed limit increases, curve radius decreases, and superelevation rate decreases. The sensitivity between the expected number of roadway departure crashes and radius is highest for smaller radii and the sensitivity continually decreases as the radii get larger and larger. Posted speed limit, superelevation rate, and curve radius are treated as an interaction term and impact the calculated CMF in tandem (e.g., the safety effect of changing the horizontal curve radius is dependent upon the speed limit and superelevation). The CMF is calculated using the following equation in figure 18.

$$CMF = \frac{exp^{\left(4.101\times\left(\frac{V_{proposed}^{2}}{15\times R_{proposed}}-0.01\times e_{proposed}\right)\right)}}{exp^{\left(4.101\times\left(\frac{V_{existing}^{2}}{15\times R_{existing}}-0.01\times e_{existing}\right)\right)}}$$

Figure 18. Equation. CMF for a combination of posted speed limit, superelevation rate, and curve radius.

Figure 19 graphically presents the CMF (for 20, 40, and 55 mph posted speed limits) for horizontal curve radius relative to the HSM CMF for horizontal curve radius. As the figure shows, the CMF increases as horizontal curve radius decreases at a given speed. Additionally, the CMF increases for a change in radius as the posted speed increases (e.g., the CMF for changing a radius at 20 mph is smaller than making the same change at 55 MPH). Figure 19 also includes the CMF for changing the horizontal curve radius from the HSM. For smaller radii, the HSM radius CMF is closer to the lower speed curves; as radius increases, the HSM radius CMF becomes closer to the higher speed curves. Figure 20 provides a graph of CMFs versus horizontal curve radius by speed limit, where the base condition is the minimum radius to maintain a normal crown (assuming the superelevation rate is 8 percent) from the *AASHTO Green Book*. The minimum radius provided for each curve is approximately 100 ft less than the minimum design radius from the *AASHTO Green Book*.

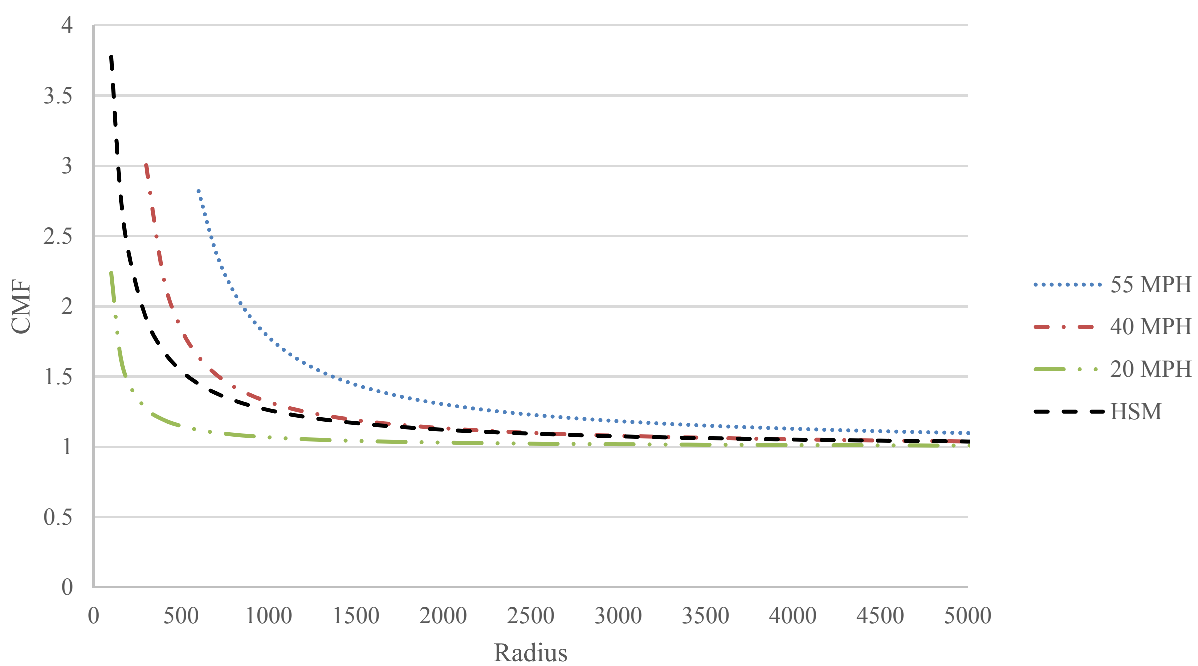

Figure 19. Graph. CMF for total crashes by speed versus HSM.

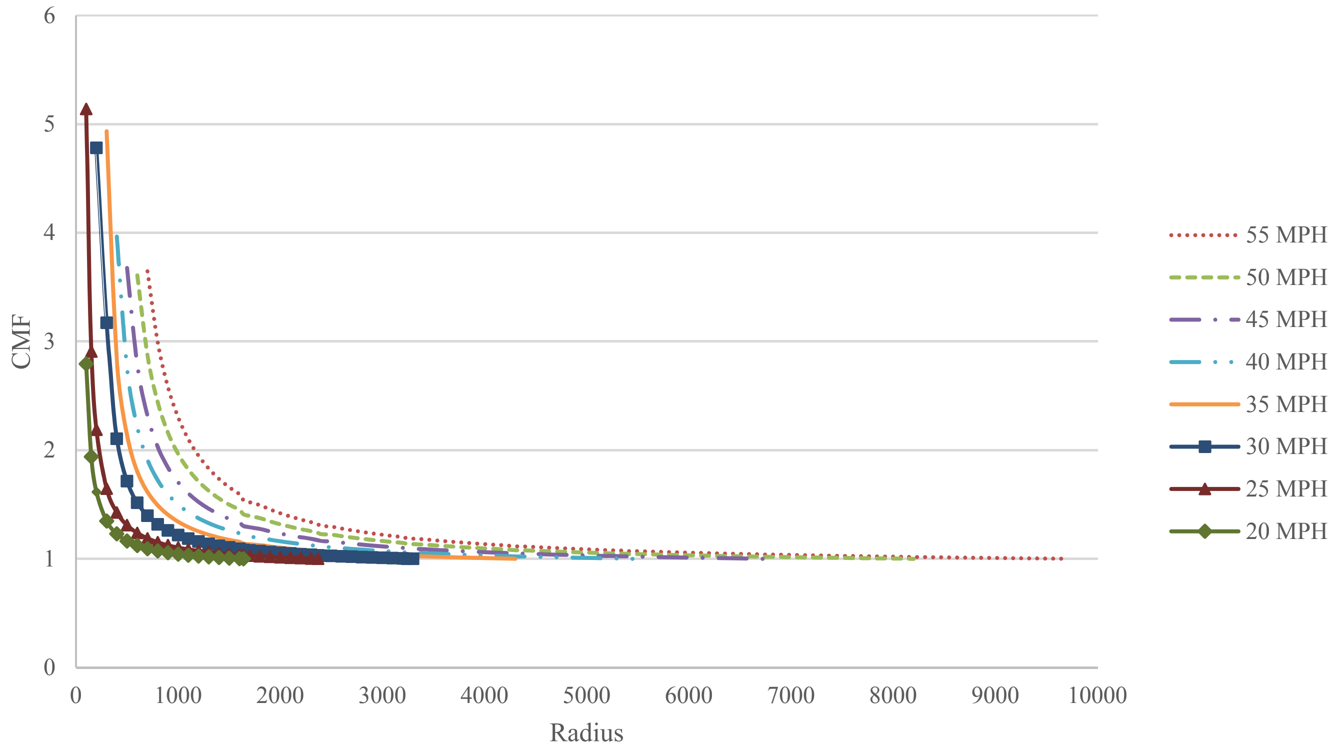

Figure 20. Graph. CMF for RwD-KABCO crashes from North Carolina radius.

The friction demand CMF has a broad range of applicability. While this document provides graphical representations for changing horizontal curve radius assuming a posted speed, the equation can be used for the following purposes:

- Assessing the safety effect of changing horizontal curve radius while maintaining the same speed.
- Assessing the safety effect of changing horizontal curve radius while changing the posted speed limit.
- Assessing the safety effect of changing the superelevation rate.
- Assessing the safety effect of increasing the posted speed limit on an existing curve (inputting information for existing horizontal curve radius and superelevation rate).
- Assessing the safety effect of simultaneously increasing the superelevation rate and the posted speed limit on an existing horizontal curve.
- Assessing the safety effect of simultaneously changing the posted speed limit, horizontal curve radius, and superelevation rate on an existing or proposed horizontal curve.

Example:

An existing curve is posted at 45 mph, has a horizontal curve radius of 550 ft, and a superelevation rate of 8 percent. The safety impact of increasing the radius to 1,000 ft is to be calculated seen in figure 21.

$$CMF = \frac{exp^{(4.101\times(\frac{45^2}{15\times 1{,}000}-0.01\times 8.0))}}{exp^{(4.101\times(\frac{45^2}{15\times 550}-0.01\times 8.0))}} = 0.64$$

Figure 21. Equation. CMF for increasing radius to 1,000 ft.

The CMF indicates an expected 36-percent reduction in the expected number of roadway departure crashes. If the posted speed limit were proposed to be increased to 55 mph in tandem with the increase in radius, the CMF would be calculated as shown in figure 22.

$$CMF = \frac{exp^{(4.101\times(\frac{55^2}{15\times 1{,}000}-0.01\times 8.0))}}{exp^{(4.101\times(\frac{45^2}{15\times 550}-0.01\times 8.0))}} = 0.84$$

Figure 22. Equation. CMF calculation based on increase in posted speed limit and increase in radius.

The CMF indicates an expected 16-percent reduction in the expected number of roadway departure crashes. If the posted speed limit were proposed to be increased to 55 mph with no change in radius the CMF would be calculated as seen in figure 23.

$$CMF = \frac{exp^{(4.101\times(\frac{55^2}{15\times 550}-0.01\times 8.0))}}{exp^{(4.101\times(\frac{45^2}{15\times 550}-0.01\times 8.0))}} = 1.64$$

Figure 23. Equation. CMF calculation for increased speed with no change in radius.

The CMF indicates an expected 64-percent increase in the expected number of roadway departure crashes.

Design Consistency Effects

The estimated regression parameters for the CRR_M were positive and statistically significant at the 95-percent confidence level. The results indicate that the expected number of roadway departure crashes and fatal and injury roadway departure crashes on a horizontal curve not only increase as a function of that curve's radius, but also as a function of the radii on the upstream and downstream curves. Flatter upstream and downstream radii result in a higher number of roadway departure crashes on the subject curve, likely because drivers enter the subject curve after becoming used to the flatter radii and higher operating speeds of the surrounding curves. Sharper upstream and downstream radii result in a lower number of roadway departure crashes on the subject curve, likely because drivers enter the subject curve after becoming used to the sharper radii and lower operating speeds of the surrounding curves. The magnitudes of the CRR-M effects were similar for both the total and fatal-plus-injury crash models.

Estimated regression parameters for the RTR were positive, but were not statistically significant at any confidence level. As indicated in table 23 and

table 24, the results for this variable were not consistent between States. Regression parameters for RTR were generally significant at the 95-percent level in Pennsylvania and were essentially zero in Indiana. In Pennsylvania, the results indicate that the expected number of roadway departure and fatal and injury roadway departure crashes increase as RTR increases. In other words, the expected number of roadway departure crashes on a horizontal curve not only increases as a function of that curve's radius, but also as a function of the upstream and downstream tangent lengths. The sensitivity between the expected number of roadway departure crashes and radius is highest for smaller radii and the sensitivity continually decreases as the radii get larger and larger. Longer upstream and downstream tangents result in a higher number of roadway departure crashes on the study curve, likely because drivers enter the study curve after becoming used to the longer tangent section of alignment and higher operating speeds. Shorter upstream and downstream tangents result in a lower number of roadway departure crashes on the study curve, likely because drivers enter the study curve without enough time to increase their speed or get used to driving on the tangent section of alignment. The fact that RTR is more related to fatal-plus-injury crashes than total crashes makes intuitive sense, as it is likely the design consistency measure most related to curve entering speeds. Additional work is needed to uncover why the RTR effect was not present in Indiana.

Other Effects

The models demonstrate several other roadway departure safety effects in addition to those from the horizontal curve and design consistency measures:

- Segment length and time period were included in the model as offset variables (i.e., setting those variables to unity provides an estimate of expected number of roadway departure crashes per mi per year).
- The models include the presence of an intersection as a covariate. In all cases, the expected number of roadway departure and fatal and injury roadway departure crashes increases when an intersection is present.
- Positive regression parameters were estimated for the absolute value of grade on the study curve. This indicates the expected number of roadway departure and fatal and injury roadway departure crashes increases when the absolute value of grade increases.
- Negative regression parameters were estimated for total lane width and total shoulder width. This indicates the expected number of roadway departure and fatal and injury roadway departure crashes decreases when total lane width or total shoulder width increases.
- Models included a State of Pennsylvania indicator variable to capture other unmeasured differences between Pennsylvania and Indiana. For all roadway departure crashes, the estimated parameter was always negative (indicating a lower number, on average, of

reported roadway departure crashes in Pennsylvania) and statistically significant at the 95-percent level. For fatal and injury roadway departure crashes, the estimated parameter was always positive (indicating a higher number, on average, of fatal and reported injury roadway departure crashes in Pennsylvania) and statistically significant at the 95-percent level. These findings are intuitive for fatal and injury roadway departure crashes due to central Pennsylvania's dynamic topography compared to Indiana's flatter topography. Furthermore, it is intuitive that Indiana would have a higher number of reported PDO roadway departure crashes due to a lower reporting threshold. The reporting threshold is $1,000 in Indiana as opposed to a disabled vehicle in Pennsylvania.

Comparison with Utah/Washington Analysis

Table 29 provides a comparison between the results from the previous analysis (Utah/Washington) and the results from this analysis (RID). The comparison shows that the confidence intervals overlap for each characteristic for each crash severity. However, the coefficients for the inverse of horizontal curve radius (for both crash severities) and for RTR (for fatal and injury roadway departure crashes) differ substantially between the analyses. It appears that the incorporation of additional variables in the model reduced the magnitude of the estimated coefficients compared to the States with no additional information available. The RID provided what appeared to be higher quality data for curve radii and superelevation than the Utah and Washington datasets, as well as additional high-quality data for more variables, likely helping to identify more representative effects of the factors of interest.

Table 29. Results comparison.

Notation	RwD-KABCO Utah	RwD-KABCO Washington	RwD-KABC Utah	RwD-KABC Washington	RwD-KABCO Indiana	RwD-KABCO Pennsylvania	RwD-KABC Indiana	RwD-KABC Pennsylvania
1/R	860.36	186.74	801.18	259.56	464.66	55.77	405.32	58.99
fd	4.292	1.474	4.573	2.047	4.101	0.464	3.346	0.496
CRR_M	0.253	0.118	0.249	0.169	0.165	0.039	0.136	0.042
RTR	0.037	0.096	0.176	0.126	0.018	0.024	0.008	0.018

CONCLUSIONS AND RECOMMENDATIONS

This research explored and estimated relationships between elements of horizontal curvature and measures of design consistency, and the expected number of roadway departure crashes along horizontal curves on rural, two-lane, two-way roads. The research explored these relationships using a negative binomial (NB) regression modeling approach and observing the regression parameter estimates corresponding to the measures. Data used for model estimation were drawn from the SHRP 2 RID 2.0. The data were originally collected for use with the NDS data but were suitable for analysis with State-maintained traffic and crash data. A few points regarding the methodology are worth noting in this context:

1. The study design was cross sectional. Whether estimated regression parameters from cross sectional studies can ever represent safety effects estimates is a matter of ongoing debate. More recent safety research has implemented cross sectional study designs intended to "mimic" randomized experiments and strengthen confidence in safety effects estimates, such as the propensity scores – potential outcomes framework. Road safety researchers have limited experience using the propensity scores – potential outcomes framework for treatments represented by continuous variables, such as alternative measures of design consistency. The scope of this effort did not support the use of the propensity scores – potential outcomes framework.
2. The characteristics of the sample of horizontal curves, specifically related to the number or proportion of larger radii curves in the sample, appears to impact the estimated association between inverse curve radius and expected number of roadway departure crashes. Since the modeling technique is non-linear, including larger radii curves will impact the coefficient estimates, which can lead to larger CMFs for small radii curves upon application. This challenge appeared to be addressed by including an interaction between inverse curve radius, speed, and superelevation, represented by side friction demand.
3. The crash data available for Indiana did not allow for elements of the RID data to be combined with the Washington and Utah data analyzed previously. The definitions of roadway departure crashes differ slightly between the two datasets. Information on harmful events was not available to fully differentiate crash types within non-collision crashes. However, non-collision crashes accounted for a very small proportion of roadway departure crashes. The Pennsylvania data could be combined with the Washington and Utah data in future analyses.

The analysis results showed that the expected number of roadway departure crashes along a horizontal curve changes not only as a function of that curve's radius, but also as a function of the radii on the upstream and downstream curves and, for Pennsylvania, the upstream and downstream tangent lengths. More specifically, for a subject curve with a given set of characteristics:

- Flatter upstream and downstream radii result in a higher number of roadway departure crashes on the subject curve, likely because drivers enter the subject curve after becoming used to the flatter radii and higher operating speeds of the surrounding curves.

- Sharper upstream and downstream radii result in a lower number of roadway departure crashes on the subject curve, likely because drivers enter the subject curve after becoming used to the sharper radii and lower operating speeds of the surrounding curves.
- Longer upstream and downstream tangents result in a higher number of roadway departure crashes on the subject curve in Pennsylvania, likely because drivers enter the subject curve after becoming used to the longer tangent section of alignment and higher operating speeds. This effect was not observed at a statistically significant level in Indiana.
- Shorter upstream and downstream tangents result in a lower number of roadway departure crashes on the subject curve in Pennsylvania, likely because drivers enter the subject curve without enough time to increase their speed or get used to driving on the tangent section of alignment. This effect was not observed at a statistically significant level in Indiana.

Such findings are intuitive given the concept of design consistency and represent a potential advancement to existing predictive methods in the HSM, which estimates the expected number of crashes on a segment as a function of the characteristics of only that segment. Future work should attempt to explore why the RTR effect did not seem to carry over to a State like Indiana, which has a much different roadside environment than Utah, Washington, or Pennsylvania.

Additionally, the analysis results showed that the expected number of roadway departure crashes on a horizontal curve changes due to the combination of that curve's radius and the presence of a normal crown. The project team attempted to further incorporate the main effect of actual superelevation rate (on superelevated curves), but did not find a statistically significant result. The results showed that the expected number of roadway departure crashes increase at a faster rate for smaller horizontal curve radii when the curve has a normal crown. Future research should further explore the direct effect of superelevation rate and should extend this analysis to analyze the safety impacts of superelevation deficiency (i.e., existing superelevation that is less than recommended by the *AASHTO Green Book*).

The analysis results did, however, show that the expected number of roadway departure crashes on a horizontal curve increases as friction demand increases. Friction demand is calculated using the point-mass model, the posted speed limit as a substitute for design speed, the horizontal curve radius, and the superelevation rate. The results show an interaction between posted speed, curve radius, and the superelevation rate.

The findings in the RID analysis are similar to those in the previous analysis of the Washington and Utah datasets. The safety effects of horizontal curve characteristics have a smaller effect in this analysis (however, the confidence intervals overlap), due to the ability to address potentially confounding characteristics with the RID dataset that was not possible with the Utah and Washington datasets. Additional safety treatments were available in the RID dataset (e.g., rumble strip presence, presence of warning signs), but these factors were more likely to be influenced by site selection bias (e.g., chevrons are typically placed on curves with a history of roadway departure crashes) or were not significant in models. Future research should expand the analysis to all applicable RID sites to bolster sample size and to allow for utilization of the propensity scores – potential outcomes framework. Additionally, since the RID contains information on roadways most likely to be traveled by NDS participants, the curves in this database are often on arterials, which have much larger curve radii and more forgiving design characteristics than other, lesser traveled roadways. This analysis would benefit from including more curves with radii closer to the limiting radius from geometric design policies and from lower AADT roadways. Finally, the NDS itself could be used to try and verify the speed and lane keeping impacts of the horizontal curve and design consistency characteristics explored in this study to further support interpretations of the findings.

CHAPTER 6. SAFETY EVALUATION OF GUIDERAIL AND CURVE DELINEATORS ON HORIZONTAL CURVE ALONG TWO-LANE RURAL HIGHWAYS

This chapter of the report considers the safety effects of several countermeasures that may be implemented along horizontal curves of two-lane rural highways. These include the following: 1) the presence of guiderail with delineators along the curve, 2) the presence of guiderail only, and 3) the presence of only delineators along the curve. The first section of the chapter provides background information related to past research that reported on the safety effects of guiderail and delineators. The second section of the chapter describes the study sites, while the third section describes the data. The fourth section of the chapter summarizes the analysis methodology, and the fifth section explains the results. The final section offers conclusions from the evaluation.

BACKGROUND

According to the Manual on Uniform Traffic Control Devices (MUTCD) (2009), delineators can be used along the roadside when there are changes in horizontal alignment, thus providing guidance and information to users. Delineators should comprise of a retroreflective device whose minimum dimension is 3 inches for both width and height. The color of the delineators shall be consistent with color of the roadway edge lines. There is no specific information in the MUTCD concerning the use of delineators on two-lane rural highways; however, the following placement guidance is provided:

- Delineators should be placed 2 to 8 ft outside of the roadway edge, unless an obstacle exists, where the placement should be converted to be in line with or inside the innermost edge of the obstacle.
- The mounting height between the bottom of the retroreflective device and the elevation of roadway near edge should be approximately 4 ft. The MUTCD recommends minimum delineator spacing, ranging from 20 to 300 ft, depending upon radius of curve. The spacing should be adjusted along the curve so that drivers can recognize multiple delineators simultaneously when traversing horizontal curves.

Delineators

The effect of post-mounted delineators (PMDs) on vehicle operating speed, driver performance and crash risk has been reported by many researchers. This section of the literature review describes the findings from studies that reported on the relationship between PMDs and driving performance, with a focus on operating speed and crash risk along horizontal curves.

Zador et al. (1987) evaluated the short-term and long-term effects of PMDs on the operating speed and lateral position of vehicles in the travel lane on horizontal curves. There were 46 study sites from Georgia and 5 study sites from New Mexico included in the field evaluation, all of which were on rural two-lane highways. A total of four independent variables were included in the evaluation. These included: treatment, direction of curve or turn, vertical alignment (grade < -2 percent or down, -2 percent ≤ grade ≤ 2 percent or level, and 2 percent < grade or up) and sharpness of curve (less or more sharp within the grade and turn class). The delineators were installed on both sides of metal posts along the outside of the horizontal curves. The type, size, location and spacing requirements for the PMDs were consistent with the MUTCD. A traffic data recorder (TDR) was used to measure the operating speed and lateral position of vehicles in the lane as they traversed the curves. Two TDRs were set up for each observation period -- one was located 100 ft before the beginning of the curve and the other was 100 ft after the beginning of that curve. Data were collected during three different time periods to evaluate the short-term and long-term effects of the PMDs. These periods were shortly before and several weeks after the PMDs were installed, and 6 months after the installation. During each period, about 100 to 150 vehicles were recorded during the daytime and nighttime travel periods separately.

The results indicated that vehicle speed and lane position were significantly influenced by PMDs, compared to untreated sites. The short-term effect demonstrated an obvious speed increase before and after the application of PMDs for both daytime and nighttime period. Also, vehicles tended to travel towards the centerline when PMDs were installed along right-hand curves. While traveling along left-hand curves, they shifted away from the centerline before entering the curves and then towards the centerline as traversing the curves, regardless of the presence of PMDs. In addition, no evidence was found to validate significant variances over time. Overall, the authors concluded that the primary effect of PMDs was helping drivers better recognize horizontal curves ahead.

Choi et al. (2015) developed crash modification factors (CMFs) of horizontal alignment elements, including PMDs. The study sites included 10 freeways in Korea. The PMDs were installed on 753 segments, and there were 2,170 segments in the reference group, with no vertical grade or horizontal curve. Safety performance functions (SPFs) were developed for each selected freeway, and an empirical Bayes (EB) method was applied to develop the CMFs. The CMF for PMDs was 1.190 with a standard error of 0.259. This suggests that PMDs are associated with an increase in the expected total crash frequency on freeway segments in Korea.

Elvik and Vaa (2004) reported CMFs of implementing PMDs on two-lane rural roads using meta-analysis approach in 2004. The CMF for all crash type with serious injury, minor injury and possible injury severity level was 1.04 with an adjusted standard error of 0.1. The CMF for all crash type with property damage only was 1.05 with an adjusted standard error of 0.07. Both results indicate that PMDs installation is associated with a potentially increase in crash frequency.

Chrysler et al. (2009) investigated the influence of PMDs on driver response to horizontal curves. The project consisted of a nighttime driving experiment on a closed course, a driver survey of curve perception, and a field evaluation of delineation treatments. Twenty volunteers participated in the nighttime driving test, which was intended to select delineation treatments used for the field observation. Two types of PMDs were selected -- one of the PMDs contained a single reflector at the top of the post (referred to as “dot” PMDs) and the other PMDs contained a retroreflective material over the full length of the post (referred to as “full” PMDs). Four sites on two-lane rural roads in Texas were selected to conduct the before-after field investigation, where no treatments were applied in the before period and one of the selected PMDs was installed in the after period. For each approach of the selected curve, operating speed and lateral position of vehicles at the curve warning sign on the tangent segment, curve entry point, and curve midpoint were collected. The analysis showed that both “dot” PMDs and “full” PMDs significantly reduced the vehicle centerline encroachment rate. The “dot” PMDs reduced the rate by 78 percent for a vehicle with 80-inch track width and 94 percent for a 61-inch width, while the “full” PMDs reduced the rate by 77 percent and 88 percent respectively. Also, both PMDs improved lane position. Drivers tended to drive about one foot closer to the edge line when compared to the baseline scenario. Finally, there was no statistically significant decrease in operating speed before and after the PMDs were installed along horizontal curves.

Nygardhs et al. (2014) studied the effect of different PMD spacing on operating speeds at night. The study was conducted on a driving simulator, where a 9 m-wide and 6 km-long roadway segment was designed. The route comprised of six horizontal curves, each having different radii and turning directions. Researchers tested seven different PMD configurations, which included current standards in Norway, Sweden, Denmark, Finland, and suggestions from an expert panel, respectively. In general, the study was carried out in a way that each of the 14 participants would view every PMDs design combination in the experimental design. Throughout the simulation, the operating speed of each participant was collected at the same frequency of 10 Hz. The mean operating speed before the curve, within the curve and after the curve, as well as the whole route, was defined as the average of the mean speed of all participants. The speed data were analyzed using the univariate analyses of variance (ANOVA). The results demonstrated that if no distracted driving task was involved in the experimental run, the mean operating speed on the whole segment remained consistent between different PMDs spacing, as long as they were installed continuously along the route. Operating speed increased when PMDs were present, relative to the condition when no PMDs were present along the route. Moreover, if PMDs were installed continuously along the road, vehicle operating speeds were affected by the spacing. Finally, operating speeds appeared to be influenced more by PMDs on large radius curves than on flat radius curves.

In summary, PMDs are intended to provide visual guidance to drivers about the presence and degree of horizontal curve. Based on published research, the CMFs associated with installing PMDs were slightly larger than one, indicating an expected increase in total crash frequency. In addition, PMDs have generally been associated with increased operating speeds when compared to horizontal curves without PMDs.

Guardrail

Guardrail has proven to be effective in reducing crash severity, especially for run-off-the-road (ROR) collisions. The focus of the review in this section is on the evaluation of guardrail effectiveness when applied along roadway shoulders.

Park et al. (2016) estimated CMFs for guardrail and other roadside barriers using both the EB and full Bayes (FB) methods. In the study, 147 freeway segments in Florida were identified as a treatment group, including 127 sites where w-beam guardrail was installed and 20 sites where concrete barriers were installed. And there were 328 segments in the reference group. The results showed that there were few differences in the CMFs developed using the EB and FB methods. If considering all severity levels (KABCO), the estimated CMFs were 1.09 for all crash types and 1.01 for ROR crash type only, while if considering KAB severity levels only, the estimated CMFs were 0.85 for all crash types and 0.75 for ROR crashes. This indicates that guardrail increases the total crash frequency and ROR crash frequency when considering all severity levels. And guardrail installation appears to be associated with a reduction in severe crashes. Moreover, by estimating CMFs for ROR crash with respect to time, it was found that CMFs were 1.05 for KABCO levels and 0.89 for KAB levels during the day, and CMFs were 0.98 and 0.53 at night, respectively. This suggests that guardrail is more effective for ROR crashes during night time than day time.

Choi et al. (2015) estimated CMFs of roadside barriers including guardrails in Korea. There were 78 treated sites and 2,170 reference segments selected from 10 freeways. The statistical results showed that guardrails had a positive influence on crash reduction, where the CMF was 0.908 with a standard error of 0.431. This indicates that guardrail installation is associated with a decrease in the expected total crash frequency on freeway segments in Korea.

Elvik and Vaa (2004) reported CMFs of new guardrail along roadway embankment for ROR crashes. The estimated CMF was 0.56 with an adjusted standard error of 0.1 for fatal crash, 0.53 with a standard error of 0.05 for injury crash, and 0.93 with a standard error of 0.31 for all crashes. This suggests that guardrail is effective in reducing crash severity, while it is not deterministic in total crash frequency reduction.

Martin et al. (2013) conducted a long-term analysis of the impact of guardrail on motorway safety in France. Crash data used in the study included fatal, injury, and property damage only crashes occurring on a French motorway network from 1996 to 2010. Run-off crashes, including the type of barrier encountered, impact position, and vehicle behavior after collision, were also considered in the evaluation. The study results showed that guardrail significantly reduced injury rates in ROR crashes. The relative risk of at least one injury in a shoulder run-off-road crash was 1.9 to 2.6 times higher for roadways without guardrail, compared with segments having roadside guardrails. Also, single-sided w-beam guardrail yielded the lowest injury rate in terms of vehicle-barrier collisions.

Ben-Bassat et al. (2011) investigated the effect of guardrail, shoulder width, and roadway geometry on vehicle operating speeds and lane position in a driving simulator study. Twenty-two participants were included in the study, 11 male and 11 female. Each participant drove three different scenarios with various element combinations. These included: 0.5 m, 1.2 m and 3.0 m shoulder widths; presence or no presence of guardrail; and curve radius and direction. The results showed that vehicle speed was higher when guardrails were installed on straight roads or right-turning curves. Meanwhile, the operating speed was not necessarily higher on left-turning curves in the presence of guardrails.

To summarize, guardrail is effective when applied along roadway shoulders. The CMFs from previous studies suggests a significant reduction in crash severity, as well as the ROR crash type. With regards to total crash frequency, confidence interval of the estimated CMFs usually contain

1.0, indicating that the presence of guardrail may not influence the expected total crash frequency when installed along roadway shoulders.

STUDY SITES AND ROADWAY INVENTORY DATA

The study sites for the present study are all horizontal curves on two-lane rural highways, in which 417 sites are located in Pennsylvania and 437 sites are located in Indiana. The analysis database was developed using the Strategic Highway Research Program (SHRP) 2 Roadway Information Database (RID). The RID database included a variety of features present along a horizontal curve on two-lane rural highways, such as:

- Curve direction;
- Curve radius, and curve length;
- Superelevation and vertical grade (percent);
- Posted speed limit (mph) and curve advisory speed (mph);
- Presence of curve warning and chevron signs;
- Curve lane width and shoulder width (ft);
- Presence of rumble strips;
- Roadside barrier types and barrier percentage within the horizontal curve (percent);
- Presence of intersections;
- Presence of lighting; and
- Traffic volume (average annual daily traffic, veh/day).

The treatments of interest, delineators and guiderail, were collected manually from a combination of Google Earth street view, Pennsylvania Department of Transportation (PennDOT) online video photologs, and the RID street view. It was assumed that the delineators and guiderail were present during the entire analysis period, so the evaluation methodology employed a cross-sectional study design.

Table 30. Summary of reference and treatment study sites.

Category	Pennsylvania	Indiana	Total
Absence of both guiderail and delineators (Reference)	247	379	626
Presence of guiderail (only) (treatment)	41	46	87
Presence of delineator post (only) (treatment)	28	10	38
Presence of guiderail with post-mounted delineators (treatment)	29	2	31
Presence of guiderail with triangle delineators (treatment)	59	0	59
Presence of guiderail with both post-mounted and triangle delineators (treatment)	13	0	13
Presence of guiderail with sum of guiderail with delineators (treatment)	101	2	103
Total	417	437	854

Table 30 shows the number of horizontal curves with guiderail and delineators among the sample of curves in Pennsylvania and Indiana. There were 626 curves that did not contain either guiderail or delineators. There were 41 and 46 curves with only guiderail in the Pennsylvania and Indiana data files, respectively. There were a total of 38 horizontal curves in the sample with delineators only – this sample is not likely to yield reliable CMFs. There were 101 horizontal curves in Pennsylvania with both delineators and guiderail, but only 2 curves with both treatments in Indiana. Therefore, only Pennsylvania data were used to estimate CMFs for the dual application of delineators and guiderail. The effect of delineators only was inferred from the CMFs derived for the dual application of delineators and guiderail, relative to the CMF for guiderail only. A summary of the available sites used in the evaluation are shown in table 31.

Table 31. Study sites of the safety evaluations.

Safety Treatment	Treatment Sites	Reference Sites	State
Guiderail with delineators	101	247	Pennsylvania
Guiderail	87	626	Pennsylvania, Indiana

DESCRIPTION OF DATA

This chapter describes the data used to estimate the safety effects of the delineator and guiderail countermeasures. Two datasets were developed to estimate the CMFs – one for guiderail with delineators and a second dataset for guiderail only. The dependent and independent variables available for the analysis are described separately for the treatment and reference group sites.

Crash data for the years 2009 to 2013 (inclusive) in both Pennsylvania and Indiana were used in the evaluation. The following dependent variables were considered when developing the CMFs:

- Total crash frequency (all severities and all crash types).
- Total number of fatal and injury (FI) crashes.
- Total number of run-off-road related crashes (all severities).
- Total number of night-time crashes (all severities).

Traffic volume data were available from the year 2011 to 2013. AADT data for the years 2010 and 2009 are extrapolated using a linear trend line between the years 2011 and 2013. The following independent variables were acquired from the RID:

- Traffic volume (average annual daily traffic, veh/day);
- Average curve length (mi);
- Curve lane shoulder width (ft);
- Horizontal curve radius, and degree of curvature;
- Posted speed limit (mph);
- Curve advisory speed (mph);
- Presence of curve arrow;
- Presence of chevron signs; and
- Presence of intersections.

During the analysis period, no treatments were installed, so the study design is a cross-sectional with-without comparison. Study sites with low traffic volumes (AADT < 100 veh/day), and roadways with posted speed limits lower than 35 mph were eliminated from the data analysis files, because nearly all of these sites were reference group locations in the guiderail with delineators dataset that were dissimilar with the treatment group sites in the same dataset. As shown in table 30, there were 101 treatment sites and 247 reference sites in the guiderail with delineators dataset. Over the 5-year analysis period, there were 505 treatment site observations and 1,235 reference site observations. Since there are very few sites with delineators present in the Indiana data, only Pennsylvania sites are included in the analysis for this evaluation. In the guiderail only dataset, there were 87 treatment sites (435 observations in 5 years) and 626 reference sites (3,130 observations in 5 years) in the combined Pennsylvania and Indiana files.

Table 32. Descriptive statistics of continuous variables in the guiderail with treatment delineators dataset.

Continuous Variables	Mean	S.D.	Minimum	Maximum
Total crashes per year	0.358	0.643	0	3
Total fatal and injury crashes per year	0.200	0.465	0	3
Total run-off-road crashes per year	0.242	0.520	0	2
Total night-time crashes per year	0.109	0.318	0	2
Average annual daily traffic (veh/day)	3,942	2,698	945	15,007
Curve length (mi)	0.189	0.111	0.029	0.745
Posted speed limit (mph)	47.228	8.553	35	55
Curve advisory speed (mph)	45.000	9.960	15	55
Difference between speed limit and advisory speed (mph)	2.327	5.703	0	25
Curve lane shoulder width (ft)	3.534	1.667	0.351	8.256
Degree of curvature	4.817	6.908	0.370	58.795

Table 33. Descriptive statistics of continuous variables with reference delineators dataset.

Continuous Variables	Mean	S.D.	Minimum	Maximum
Total crashes per year	0.272	0.584	0	4
Total fatal and injury crashes per year	0.139	0.386	0	3
Total run-off-road crashes per year	0.169	0.441	0	3
Total night-time crashes per year	0.101	0.335	0	3
Average annual daily traffic (veh/day)	3,675	2,289	291	12,665
Curve length (mi)	0.159	0.074	0.038	0.400
Posted speed limit (mph)	45.972	7.955	35	55
Curve advisory speed (mph)	44.595	8.822	20	55
Difference between speed limit and advisory speed (mph)	1.457	4.039	0	20
Curve lane shoulder width (ft)	2.743	1.829	0	10.595
Degree of curvature	3.497	3.824	0.353	25.649

Table 34. Descriptive statistics of categorical variables with treatment delineators dataset.

Categorical Variables	Proportion for Yes Category	Proportion for No Category
Presence of curve arrow or chevron signs	7.92%	92.08%
Presence of intersection in the curve	48.51%	51.49%
Curve lane shoulder width under 4 ft	48.51%	51.49%
Difference between speed limit and advisory speed is above 0	17.82%	82.18%

Table 35. Descriptive statistics of categorical variables with reference delineators dataset.

Categorical Variables	Proportion for Yes Category	Proportion for No Category
Presence of curve arrow or chevron signs	5.67%	94.33%
Presence of intersection in the curve	60.32%	39.68%
Curve lane shoulder width under 4 ft	60.32%	39.68%
Difference between speed limit and advisory speed is above 0	13.77%	86.23%

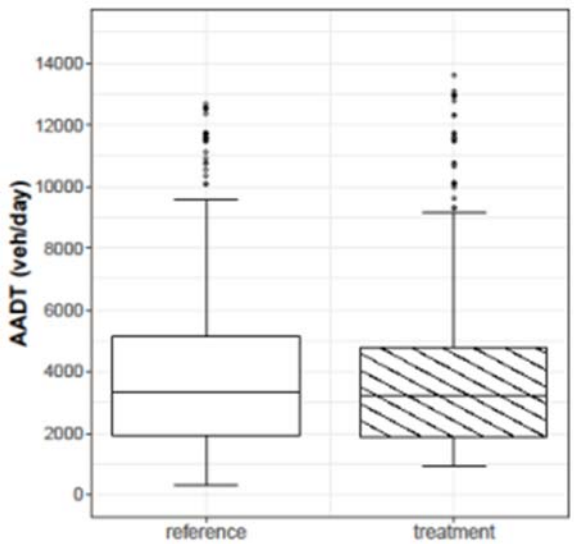

Figure 24. Graph. Box plot of AADT in the delineator with guiderail dataset.

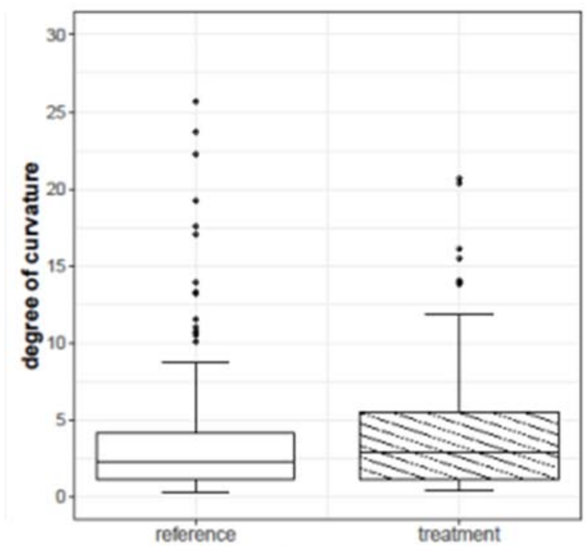

Figure 25. Graph. Box plot of degree of curvature in the delineator with guiderail dataset.*

Note: * represents box plot of degree of curvature excluding one outlier larger than 30.

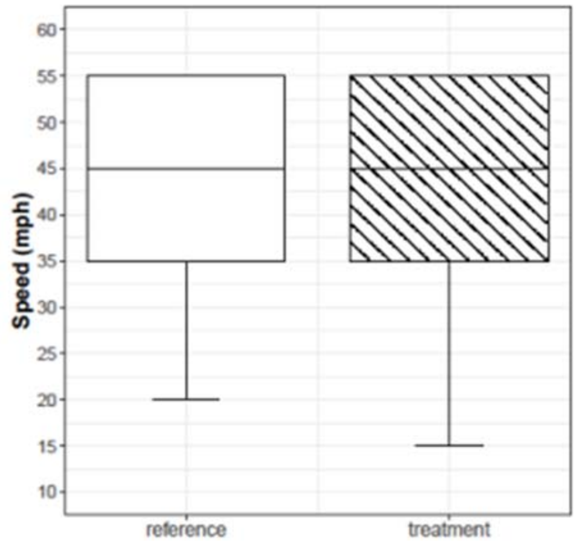

Figure 26. Graph. Box plot of curve advisory speed in the delineator with guiderail dataset.

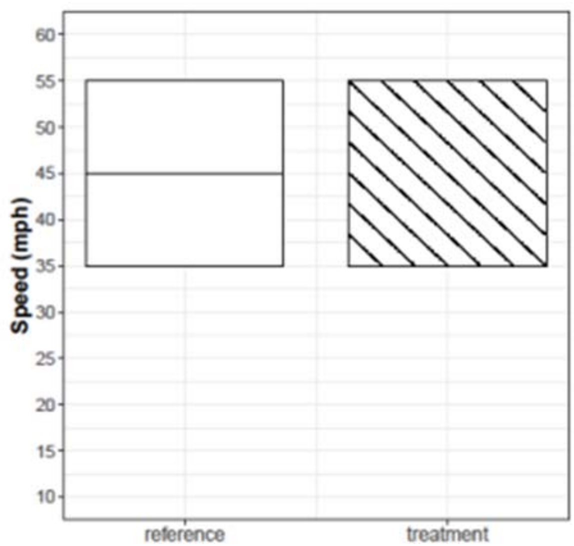

Figure 27. Graph. Box plot of posted speed limit in the delineator with guiderail dataset.

Table 32, table 33, table 34, and table 35 show summary statistics of the dependent and independent variables in the treatment and reference groups in the guiderail with delineator dataset. The average crash frequency of all crash types is higher in the treatment group than in the reference group. The mean nighttime crash frequencies are nearly identical between the two groups. Table 32, table 33, table 34, and table 35 also show that most of the roadway characteristics are similar between the treatment and reference groups. Figure 24, figure 25, figure 26, and figure 27 show box plots of AADT, degree of curvature, posted speed limit, and the curve advisory speed. With respect to the key features of the curve, the treatment sites and reference sites are comparable.

Table 36. Descriptive statistics of continuous variables in the guiderail only dataset.

Continuous Variables	Mean	S.D.	Minimum	Maximum
Total crashes per year (treatment)	0.349	0.835	0	9
Total crashes per year (reference)	0.484	1.321	0	28
Total fatal and injury crashes per year (treatment)	0.122	0.386	0	4
Total fatal and injury crashes per year (reference)	0.153	0.438	0	4
Total run-off-road crashes per year (treatment)	0.239	0.660	0	7
Total run-off-road crashes per year (reference)	0.286	0.695	0	6
Total nighttime crashes per year (treatment)	0.168	0.532	0	7
Total nighttime crashes per year (reference)	0.189	0.543	0	6
Average annual daily traffic (veh/day) (treatment)	4,757	2,801	703	16,808
Average annual daily traffic (veh/day) (reference)	4,341	2,747	153	27,331
Curve length (mi) (treatment)	0.199	0.110	0.045	0.844
Curve length (mi) (reference)	0.188	0.099	0.031	0.768
Posted speed limit (mph) (treatment)	50.632	6.564	35	55
Posted speed limit (mph) (reference)	49.353	6.948	35	55
Curve advisory speed (mph) (treatment)	48.736	8.459	15	55
Curve advisory speed (mph) (reference)	47.764	8.746	15	55
Difference between speed limit and advisory speed (mph) (treatment)	2.011	5.338	0	30
Difference between speed limit and advisory speed (mph) (reference)	1.621	4.818	0	40
Curve lane shoulder width (ft) (treatment)	4.313	2.629	0	11
Curve lane shoulder width (ft) (reference)	2.858	2.529	0	11
Degree of curvature (treatment)	4.098	4.049	0.442	27.828
Degree of curvature (reference)	3.827	4.836	0.352	55.655

Table 37. Descriptive statistics of categorical variables in the guiderail only dataset.

Categorical Variable	Proportion Yes Category	Proportion No Category
Presence of curve arrow or chevron signs (treatment)	6.90%	93.10%
Presence of curve arrow or chevron signs (reference)	6.07%	93.93%
Presence of intersection in the curve (treatment)	52.87%	47.13%
Presence of intersection in the curve (reference)	49.68%	50.32%
Curve lane shoulder width under 4 ft (treatment)	48.28%	51.72%
Curve lane shoulder width under 4 ft (reference)	81.79%	18.21%
Difference between speed limit and advisory speed is above 0 (treatment)	14.94%	85.06%
Difference between speed limit and advisory speed is above 0 (reference)	13.42%	86.58%

Table 36 and table 37 show summary statistics for the dependent and independent variables of the treatment and reference groups in the guiderail only dataset. The average crash frequency for all crash types in the treatment group is lower than those in the reference group. Many roadway characteristics, such as AADT, curve length, posted speed limit, advisory speed, degree of curvature, presence of curve arrows or chevron signs, and the presence of intersections, are nearly identical between the two groups. The lane width and shoulder widths are higher in the treatment group than in the reference group. Figure 28, Figure 29, figure 30, and Figure 31 shows box plots of AADT, degree of curvature, posted speed limit, and the curve advisory speed. With respect to the key features of the curve, the treatment sites and reference sites are comparable.

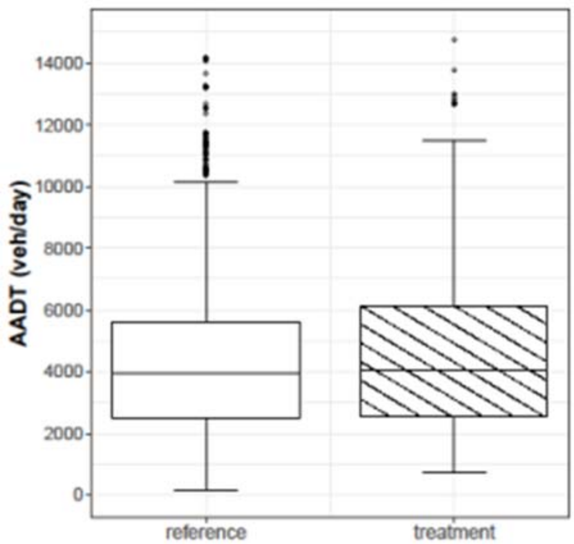

Figure 28. Graph. Box plot of AADT in the guiderail dataset.

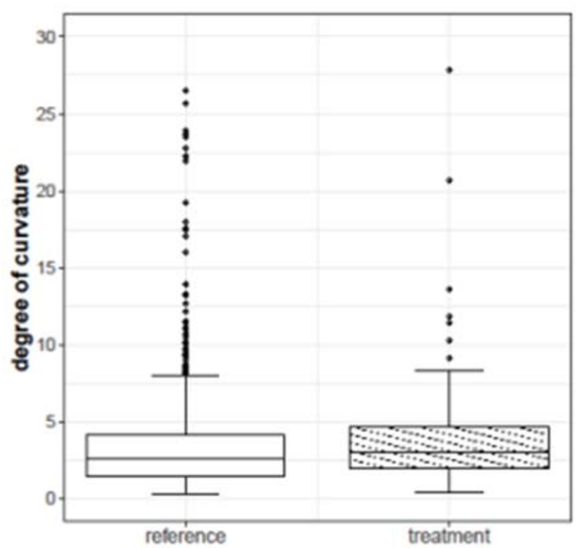

Figure 29. Graph. Box plot of degree of curvature in the guiderail dataset.*

Note: * represents box plot of degree of curvature excluding one outlier larger than 30.

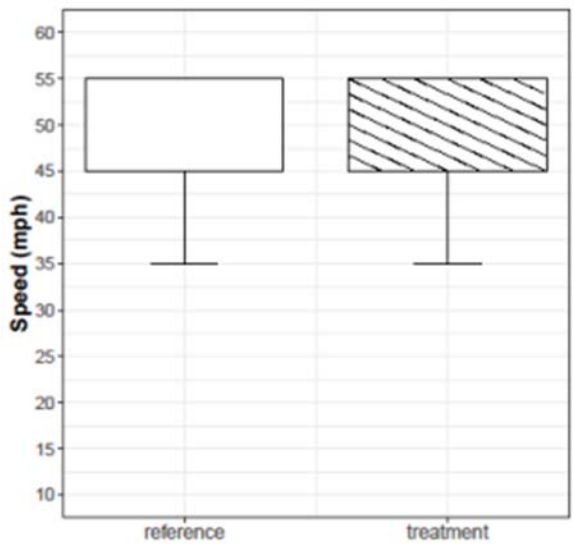

Figure 30. Graph. Box plot of posted speed limit in the guiderail dataset.

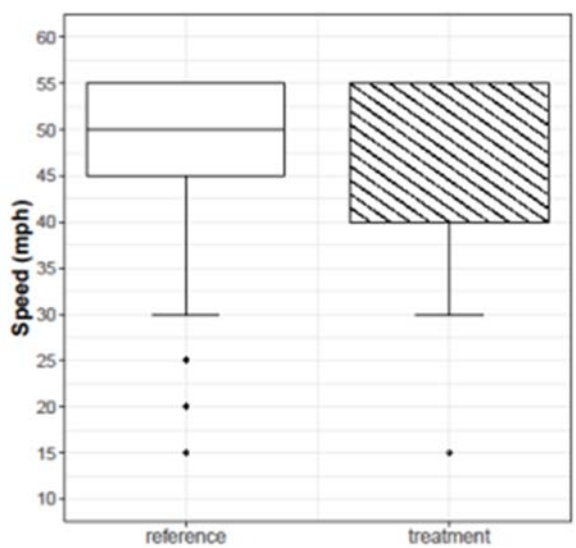

Figure 31. Graph. Box plot of curve advisory speed in guiderail dataset.

METHODOLOGY

The propensity scores-potential outcomes framework was used in the present study to estimate the safety effects of guiderail with delineators and guiderail only. This section of the report provides an overview of the propensity score-potential outcome framework, including the basic structure of the counterfactual framework, estimation of propensity scores, matching algorithms, matching considerations, and methods to estimate treatment effects for safety countermeasures.

Propensity Scores-Potential Outcomes Framework

The propensity score-potential outcomes framework is a causal analysis method that incorporates propensity score matching into treatment effect estimation. It is an advanced quasi-experimental study design methodology. The goal of this framework is to estimate the treatment effect using a subset in which the entities without the treatment are comparable to the treatment. This methodology is applicable to non-randomized observational data and removes several biases associated with cross-sectional methods (e.g. treatment selection bias, and bias due to confounding variables). The propensity scores are often estimated using a logit model. Then, the treated and untreated entities are matched according to the estimated propensity score. At last, non-parametric or parametric statistical methods can be used to estimate the treatment effect of a safety countermeasure.

Counterfactual Framework

The counterfactual framework emphasizes that an entity selected into either the treatment or no-treatment group have potential outcomes in both States, including the observed and unobserved outcomes. (Rosenbaum 2002) The counterfactual framework is illustrated by the following equation seen in figure 32.

$$Y_i = \pi_i {Y_i}^T + (1 - \pi_i){Y_i}^U$$

Figure 32. Equation. Counterfactual framework.

Where Y_i is the outcome for entity *i*; Y_i^T is the outcome for treated entity *i*; Y_i^U is the outcome for untreated entity *i*; π_i is the probability of entity *i* receiving a treatment.

The core of the counterfactual framework is the potential outcomes. Considering that the potential outcome *Y* is associated with a range of causal states (treated *T or* untreated *U),* for each unit *i* actually experiencing the treatment, the treated outcome $Y^T{}_i$ is associated with causal state *T*. This outcome is actually observed. The potential outcome is the untreated outcome $Y^U{}_i$ of the same unit *i* and is associated with another causal state *U* and is thus hypothetical (counterfactual). The goal of causal analysis in the potential outcomes framework is to learn the difference in outcomes of a unit *i* while it experienced treatment and the same unit *i* while it did not experience the treatment (or experienced other treatments). This comparison is counterfactual by necessity because it involves the comparison between observable events $Y^T{}_i$ and the unobservable events $Y^U{}_i$. In the estimation of a treatment effect, given a unit *i* and a treatment action *T*, each action-unit pair is a potential outcome. The potential outcome framework is illustrated in table 38.

Table 38. Potential outcomes framework.

Status	Treated	Untreated
Treated	Observed ($Y^T{}_i \mid T_i = 1$)	Counterfactual ($Y^T{}_i \mid T_i = 0$)
Untreated	Counterfactual ($Y^U{}_i \mid T_i = 1$)	Observed ($Y^U{}_i \mid T_i = 0$)

The treatment effect can be considered as either additive or multiplicative. In traffic safety research, since the variables in crash count models usually having multiplicative effects on the outcome (e.g., CMFs), the treatment effects are typically considered multiplicative. The multiplicative treatment effect is defined as the outcome of the treated divided by the outcome of the untreated. There are several treatment effect estimates of interest in transportation safety research such as the average treatment effect (ATE) and the average treatment effect for the treated (ATT). In traffic safety study, ATT is easier to identify and theoretically more informative since it describes the impact of treatment T for the units (horizontal curves) who were actually exposed to it. The parameters in the outcome model generally indicate the ATT. ATT is estimated as shown in the following equation seen in figure 33.

$$\Delta_{ATT} = \frac{\frac{\sum_{i=1}^{m}\left(Y_i{}^T \middle| T_i = 1\right)}{m}}{\frac{\sum_{i=1}^{m}\left(Y_i{}^T \middle| T_i = 0\right)}{m}} = \frac{\sum_{i=1}^{m}\left(Y_i{}^T \middle| T_i = 1\right)}{\sum_{i=1}^{m}\left(Y_i{}^T \middle| T_i = 0\right)}$$

Figure 33. Equation. Estimate of average treatment effect of treated.

Where Δ_{ATT} is the average treatment effect for treated; Y_i^T is the outcome for treated entity i; T_i is the treated state of entity i; m is the number of treated entities.

The third estimate is the average treatment effect for the untreated (ATU). It estimates the average treatment effect for the untreated entities if they received treatment. It is useful when researchers would like to know what would have been the outcome for the untreated entities if the treatment had been applied. ATU is defined as seen in figure 34.

$$\Delta_{ATU} = \frac{\frac{\sum_{i=1}^{m}\left(Y_i{}^U \middle| T_i = 1\right)}{s}}{\frac{\sum_{i=1}^{m}\left(Y_i{}^U \middle| T_i = 0\right)}{s}} = \frac{\sum_{i=1}^{s}\left(Y_i{}^U \middle| T_i = 1\right)}{\sum_{i=1}^{s}\left(Y_i{}^U \middle| T_i = 0\right)}$$

Figure 34. Equation. Estimate of average treatment of untreated.

Where Δ_{ATU} is the average treatment effect; Y_i^U is the outcome for untreated entity i; T_i is the treated state of entity i; and s is the number of untreated entities.

In traffic safety research, Δ_{ATE} and Δ_{ATT} are typically the two parameters of interest. ATT is usually preferred when evaluating the effect of a safety countermeasure since it is based on data from a sample of sites included in the evaluation.

Estimating Propensity Scores

Propensity scores for the treatment are often estimated using a binary logit regression model. (Shenyang Guo 2010). The binary logit model is specified in the following equation in figure 35.

$$p(x_i) = Pr\,(T_i = 1|\,X_i = x_i)$$

$$logit\,(p) = log\left(\frac{p}{1-p}\right) = \sum \beta_i x_i + \beta_0$$

Figure 35. Equation. Propensity score logit model.

Where $p(x_i)$ is the propensity of entity i being treated given the vector of covariates x_i; p is the propensity score; x_i is the vector of covariates; β_0, and β_i are the coefficients to be estimated in the logit model.

Matching Algorithms

There are several matching algorithms that could be used to match treated and untreated entities based on the propensity scores. The algorithms can be categorized into greedy and optimal matching. (Shenyang Guo 2010) Greedy matching is a linear matching algorithm. It works sequentially to find the best match (e.g., closest propensity score) for one treatment unit before creating a match for the second treatment unit. Once a match is created, the treatment unit is removed from further consideration. If replacement is allowed, a single control unit can be copied and matched to multiple treatment units. If matching is done without replacement, the control unit will be removed from further consideration. Greedy matching techniques include algorithms such as nearest-neighbor (NN) and Mahalanobis matching. Greedy matching divides large decision-making problems into a series of single decisions and processes them without reconsideration. Optimal matching algorithms create, break, and rearrange the matches in order to produce the minimum overall sum of match distances. They are basically optimization methods based on network flow theory. (Bertsekas 1991, Rosenbaum 2002, Shenyang Guo 2010) Optimal matching can be done as full matching or pair matching.

In this study, NN matching is used for several reasons. First, it is an effective algorithm to find good matches. It produces the best possible matches from a pair is chosen with the closest propensity scores. Second, NN matching is more efficient than optimal matching algorithms due to its linear decision path. Optimal matching usually requires higher computational time since the matched pairs are reconsidered in the optimization process (Holmes 2013). Finally, NN matching is a flexible algorithm to accommodate different sample sizes in the treatment or reference group by adjusting the matching ratio and replacement setting.

Matching Considerations

Overlap and Common Support

After estimating the propensity scores, an important step is to check the overlap and common support of the propensity score distributions between the treatment and control groups. Sufficient common support is necessary to process the matching; otherwise, a lack of common support indicates that the treatment and control groups are too dissimilar for comparison.

Variable Selection

The goal of the propensity score model is to balance the confounding covariates, thus variables should be carefully selected in the model. Rubin and Thomas (1996) and Brookhart et al. (2006) indicated that, in order to minimize the variance but not generate extra bias, the covariate set should include all of the variables related to both the treatment assignment and the outcome, as well as the variables that are not significantly associated with the treatment assignment but associated with the outcome. At the same time, variables that may have been affected by the treatment should not be included in the model. (Rosenbaum 1984) The judgement of variable selection requires careful consideration since excluding an important confounder will increase the bias.

Matching ratio

NN matching can be applied on a 1:1 or 1: k basis, and "with replacement" or "without replacement." 1:1 matching discards a large number of observations but avoids poor matches, while 1: k matching produces higher precision, but sometimes does not decrease bias. When treatment and control groups have similar sample size, and better matches are preferred, 1:1 matching is recommended. (Holmes 2013, Start 2010)

Replacement

Replacement is defined as allowing a control entity to be matched with multiple treatment entities. Matching with replacement allows a control entity to be reconsidered after pairing it with a treatment entity. Replacement is usually applied when the treatment entities significantly outnumber the control entities in the sample. In order to keep as many observations as possible, and reduce the bias and variance, it is necessary to allow replacement. On the other hand, when the sample size in the control group is large enough and there is enough common overlap, matching without replacement is preferred. (Caliendo and Kopeinig 2008, Shenyang Guo 2010, Stuart 2010)

Caliper Width

Caliper width is the level of tolerance for the maximum distance when matching the treatment entity with the control entity. Caliper width can be applied in the matching algorithms as a common support condition. It restrains the maximum distance of the propensity score (or other distance measure) for all of the matched pairs, and thus will help avoid bad matches. There is a trade-off in specifying caliper width since larger caliper widths may lead to bias while smaller caliper widths reduce the number of observations to be used to estimate the potential outcomes. (Holmes 2013) In a large traffic safety dataset, it was recently shown that the treatment effect estimate does not change much as the caliper width changes. (Sasidharan and Donnell 2014) The recommended caliper size is about 0.20 ~ 0.25 times the standard deviation of the estimated propensity score. (Rosenbaum and Rubin 1985, Austin 2011)

Assessing Covariate Balance

Assessing covariate balance is necessary after matching. There are several ways to assess covariate balance. One method is to use the standardized bias (SB). (Rosenbaum and Rubin 1985) When the absolute difference of SB is less than 20 (absolute standardized mean difference less than 0.2), it indicates that there is no statistical difference between the treatment and control groups. (Holmes 2013) Otherwise, the Kolmogorov-Smirnov (K-S) test, graphical checks such as box-plots or quantile-quantile (QQ) plots, can be used to check the balance of continuous variables. (Sekhon and Grieve 2008, Franklin et al. 2014, Garrido et al. 2014, Linden 2015)

Estimating Treatment Effect

The outcome model used to estimate the treatment effect is a negative binominal regression model (NB model). The model estimates expected crash frequency (number of crashes per year) on a roadway segment or intersection, as a function of the treatment variable and covariates. The negative binomial (NB) model is a common approach to model crash outcomes since it accounts for overdispersion that often exists in crash data. (Lord and Mannering 2010) The general functional form of a NB model is presented as follows in figure 36.

$$ln\lambda_i = \beta_t T + \beta_i X_i + \varepsilon_i$$

Figure 36. Equation. NB regression model.

Where λ_i is the expected crash frequency on roadway segment; β_t is the parameter of estimable regression parameters of treatment; β_i is the vector of estimable parameters of covariates; X_i is the vector of covariates; ε_i is the gamma-distributed error term. The mean-variance relationship for the negative binominal distribution is shown in figure 37.

$$Var(y_i) = E(y_i)[1 + \alpha E(y_i)]$$

Figure 37. Equation. Variance of NB distribution.

Where *Var(yi)* is the variance of crash outcome; *E(yi)* is the expected crash frequency on roadway segment i; α is the overdispersion parameter. Maximum likelihood estimation (MLE) is used to estimate the model parameters.

ANALYSIS RESULTS

This section presents the propensity score matching analysis for the safety effect of two countermeasures: (1) guiderail with delineators (GD), and (2) guiderail only (G only). Each analysis includes propensity score estimation results, matching outcomes, covariate balance assessment, and CMF estimation. In addition, a disaggregate analysis using the degree of curvature is conducted for both countermeasures. Finally, CMFs for delineators only are estimated using the results of the guiderail with delineators and the guiderail only results.

Safety Effects of Guiderail with Delineators (GD)

A standard NB regression model is first estimated (see table 39, table 40, table 41, and table 42) for all the crash types using the data before matching. This model is estimated in order to reveal the factors that are statistically significant for each crash type. These factors are then considered as potential covariates in the matching process. The variables included in the model indicate the association between expected total crash frequency and the covariates in the model. Negative coefficients indicate a decrease in the expected crash frequency when those features are present along a horizontal curve. The model coefficients indicate that the presence of guiderail with delineators is expected to reduce total and nighttime crash frequency, but increase fatal and injury crashes and run-off-road-related crashes. However, those estimates are produced from the unbalanced raw data, which may introduce selection bias.

Table 39. Statistical output of the cross-sectional NB regression for total crashes (GD).

Variable	Coef.	S.E.
Constant	-4.851	0.709
Natural logarithm of AADT	0.612	0.084
Presence of guiderail with delineators (1 if present; 0 otherwise)*	-0.003	0.104
Degree of curvature	0.070	0.009
Presence of speed difference above 0 (1 if present; 0 otherwise)	0.372	0.140
Presence of intersection in the curve (1 if present; 0 otherwise)	0.182	0.101
Natural logarithm of length (offset)	1.000	n/a

Note: n/a not applicable; * represents the treatment variable is not statistically significant.

Table 40. Statistical output of the cross-sectional NB regression for fatal and injury (GD).

Variable	Coef.	S.E.
Constant	-5.705	0.928
Natural logarithm of AADT	0.646	0.110
Presence of guiderail with delineators (1 if present; 0 otherwise)*	0.058	0.131
Degree of curvature	0.061	0.011
Presence of speed difference above 0 (1 if present; 0 otherwise)	0.641	0.168
Presence of intersection in the curve (1 if present; 0 otherwise)	--	--
Natural logarithm of length (offset)	1.000	n/a

Note: --means the variable is not statistically significant; n/a not applicable; * represents the treatment variable is not statistically significant.

Table 41. Statistical output of the cross-sectional NB regression for ROR crash (GD).

Variable	Coef.	S.E.
Constant	-3.236	0.837
Natural logarithm of AADT	0.368	0.100
Presence of guiderail with delineators (1 if present; 0 otherwise)*	0.022	0.128
Degree of curvature	0.073	0.011
Presence of speed difference above 0 (1 if present; 0 otherwise)	0.658	0.159
Presence of intersection in the curve (1 if present; 0 otherwise)	--	--
Natural logarithm of length (offset)	1.000	n/a

Note: -- means the variable is not statistically significant; n/a not applicable; all other variables included are significant at 95-percent confidence level; * represents the treatment variable is not significant.

Table 42. Statistical output of the cross-sectional NB regression for nighttime crash (GD).

Variable	Coef.	S.E.
Constant	-4.377	1.083
Natural logarithm of AADT	0.454	0.129
Presence of guiderail with delineators (1 if present; 0 otherwise)*	-0.232	0.171
Degree of curvature	0.075	0.012
Presence of speed difference above 0 (1 if present; 0 otherwise)	--	--
Presence of intersection in the curve (1 if present; 0 otherwise)	--	--
Natural logarithm of length (offset)	1.000	n/a

Note: “--" means the variable is not significant thus excluded from the model; n/a not applicable; all other variables included are significant at 95-percent confidence level; * represents the treatment variable is not statistically significant.

Propensity score matching

Table 43 shows the propensity score matching configuration. The matching distance is defined by the propensity score, which is the probability of a horizontal curve receiving the treatment. In order to yield quality matches, 1:1 nearest neighbor matching is applied with a caliper width of 0.2. Considering the relatively sample size in the treatment and reference group, matching with replacement is applied. The statistical output for the estimated propensity score model is presented in table 44. Covariates included in the model are the variables related to both the crash outcome and treatment assignment.

Table 43. Propensity score matching configuration (GD).

Matching configuration	Matching configuration
Distance	Propensity score
Algorithm	Nearest neighbor matching
Ratio	1:1
Replacement	Without replacement
Caliper width	0.2

Table 44. Statistical output of propensity score model estimation (GD).

Variable	Coef.	S.E.	Z-statistic	P-value
Constant	0.674	0.796	0.850	0.397
Natural logarithm of AADT	0.066	0.091	0.720	0.469
Degree of curvature	0.153	0.019	7.860	<0.001
Lane shoulder width under 4 ft (1 if present; 0 otherwise)	-0.654	0.134	-4.880	<0.001
Presence of chevron signs or curve arrow (1 if present; 0 otherwise)	-0.888	0.305	-2.910	0.004
Presence of speed difference above 0 (1 if present; 0 otherwise)	0.317	0.164	1.930	0.053
Presence of intersection in the curve (1 if present; 0 otherwise)	-0.578	0.113	-5.110	<0.001
Natural logarithm of curve length	1.000	offset	n/a	n/a

Note: Wald chi-square(6) = 148.60; Log-likelihood at convergence = -955.163; n/a not applicable.

There are 419 out of 505 observations in the treatment group that are matched with 419 observations in the control group. A total of 86 treatment observations were unmatched and discarded since there are no observations within the caliper width in the reference group. Table 45 and table 46 show the summary statistics before and after matching in each group. Figure 38 and figure 39 shows the distribution of the propensity scores in the treatment and reference groups before and after matching. The distribution of propensity scores before matching differs between the treatment and reference groups. Because there is large region of common support, most of the data could be matched using the propensity scores.

Table 45. Summary of variables before matching (GD).

Variables	Mean	S.D.
Propensity score (treatment)	0.373	0.186
Propensity score (reference)	0.257	0.123
ln(AADT)	8.066	0.658
ln(AADT)	7.981	0.742
ln(length)	-1.819	0.551
ln(length)	-1.949	0.490
Degree of curvature	4.817	6.908
Degree of curvature	3.498	3.824
Curve shoulder width larger than 4 ft	0.654	0.476
Curve shoulder width larger than 4 ft	0.814	0.390
Presence of chevron signs or curve arrows	0.079	0.270
Presence of chevron signs or curve arrows	0.057	0.231
Speed difference larger than 0	0.178	0.383
Speed difference larger than 0	0.138	0.345
Presence of intersection (treatment)	0.485	0.500
Presence of intersection (reference)	0.603	0.489

Table 46. Summary of variables after matching (GD).

Variables	Mean	S.D.
Propensity score (treatment)	0.313	0.130
Propensity score (reference)	0.312	0.130
ln(AADT) (treatment)	8.049	0.647
ln(AADT) (reference)	8.066	0.755
Degree of curvature (treatment)	3.632	3.094
Degree of curvature (reference)	3.361	3.820
Curve shoulder width larger than 4 ft (treatment)	0.716	0.451
Curve shoulder width larger than 4 ft (reference)	0.699	0.459
Presence of chevron signs or curve arrows (treatment)	0.237	0.048
Presence of chevron signs or curve arrows (reference)	0.048	0.214
Speed difference larger than 0 (treatment)	0.136	0.343
Speed difference larger than 0 (reference)	0.143	0.351
Presence of intersection (treatment)	0.511	0.500
Presence of intersection (reference)	0.530	0.500

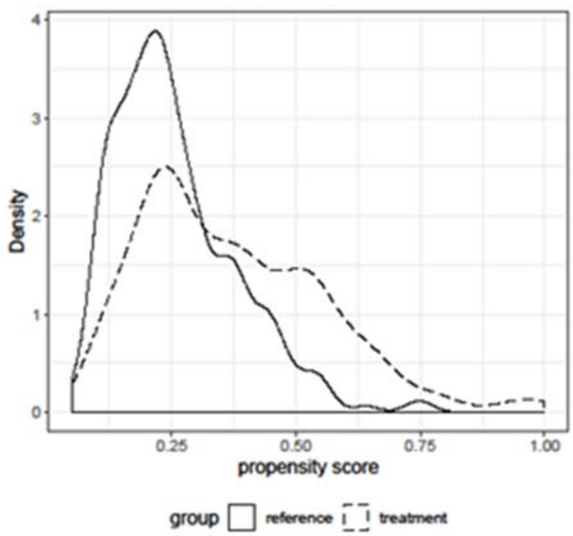

Figure 38. Graph. Propensity score distributions before matching (GD).

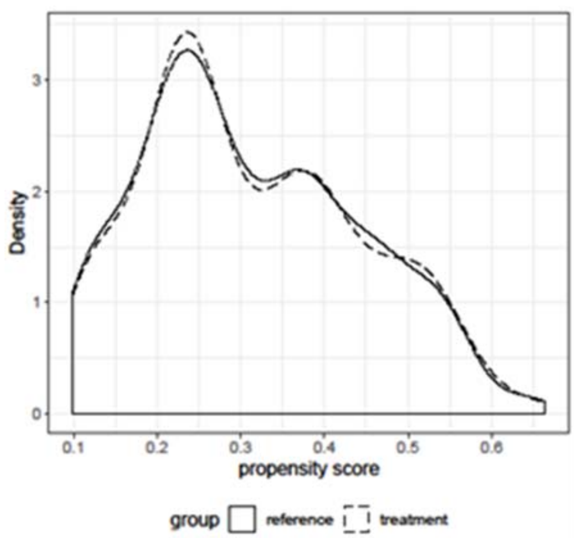

Figure 39. Graph. Propensity score distributions after matching (GD).

Covariate Balance Assessment

Table 47 shows the mean and absolute standardized mean difference (ASMD) of the propensity scores and each covariate before and after matching. An ASMD value closer to 0 indicates better balance of the covariates between the treatment and control groups. An ASMD larger than 0.20 indicates a statistically significant difference between the treatment and reference groups. As shown in table 47, the ASMD of the propensity score is reduced by 99.4 percent, from 0.738 to 0.006. The average ASMD is reduced from 0.202 to 0.043 as a result of matching. With respect to the covariates, curve length, degree of curvature, and the indicator variable for intersection presence are unbalanced between the treatment and reference groups before matching (ASMD > 0.20). However, matching produced well-balanced propensity scores. In addition, the last column of table 47 shows that all of the other covariates are improved significantly. The ASMD comparison of each covariate before and after matching is presented in Figure 40. Compared to the before-matching data, all of the ASMD values are close to zero after matching, indicating that the covariates in the treatment and reference groups are similar after matching.

Table 47. Mean and ASMD before and after matching (GD).

Treatment Variable	Mean Difference Before	Mean Difference After	ASMD Before	ASMD After	Percent Change in Balance
Propensity score*	0.116	0.001	0.738	0.006	99.4%
ln(AADT)	0.086	-0.017	0.122	0.024	80.2%
ln(length)	0.130	-0.024	0.250	0.049	81.3%
Degree of curvature	1.319	0.271	0.236	0.078	79.4%
Curve shoulder width larger than 4 ft	-0.160	0.017	0.368	0.037	89.6%
Presence of chevron signs or curve arrows	0.023	0.012	0.090	0.053	47.0%
Speed difference larger than 0	0.041	-0.007	0.111	0.021	82.4%
Presence of intersection	-0.118	-0.019	0.239	0.038	83.8%
Mean	n/a	n/a	0.202	0.043	n/a

Note: * represents the probability of receiving the treatment. All other variables are covariates and n/a represents not applicable.

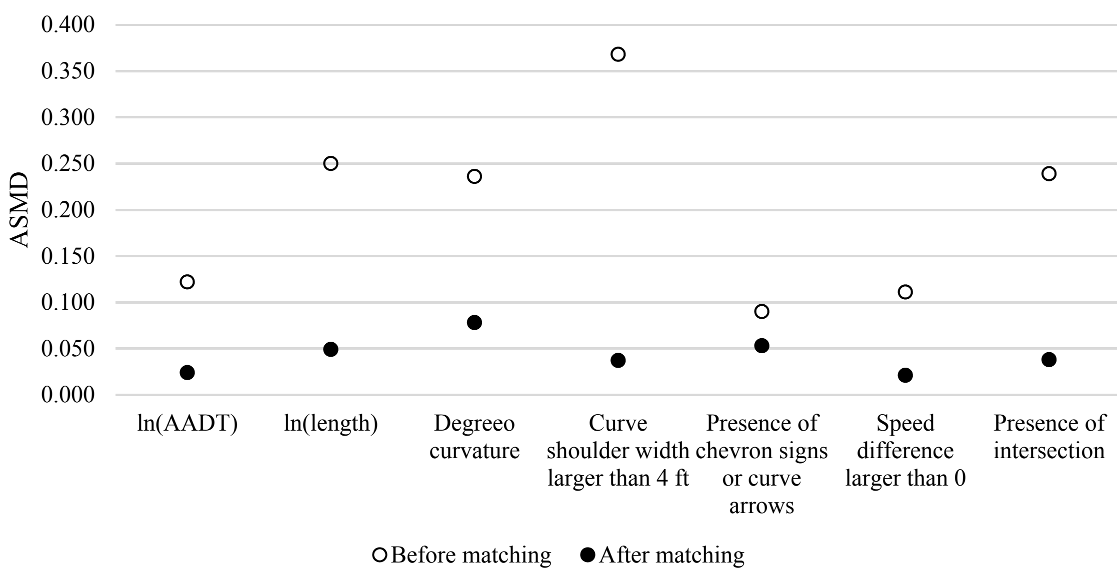

Figure 40. Graph. ASMD before and after matching (GD).

CMF Estimates

Table 48, table 49, note: n/a is not applicable; * represents the treatment variable is not statistically significant; all other variables included are significant at 95-percent confidence level.

table 50, Table 51, and Table 52 show the statistical output from a cross-sectional NB model after matching, and the resulting CMF estimates of each crash type for the guiderail with delineator dataset. The total crash, fatal and injury crash, and night-time crash frequency CMF point estimates are smaller than 1.0, which indicates that guiderail with delineators offers a safety benefit along horizontal curves of two-lane rural highways. For the run-off-road-related crash type, the presence of guiderail with delineators has a negative safety effect (CMF > 1.0). However, considering the large standard error of the treatment variable coefficient in the crash type models, and the fact that all of the 95-percent confidence intervals for all CMFs contain the value 1.0, the safety effects of guiderail with delineators along horizontal curves should be evaluated further using a larger data sample.

Table 48. Statistical output of NB model estimation after matching (GD).

Variable	Coef.	S.E.
Constant	-6.385	0.931
Natural logarithm of AADT	0.806	0.108
Presence of guiderail with delineators (1 if present; 0 otherwise)*	-0.030	0.124
Degree of curvature	0.099	0.017
Presence of speed difference above 0 (1 if present; 0 otherwise)	0.391	0.179
Natural logarithm of length (offset)	1.000	n/a

Note: n/a means not applicable; * represents the treatment variable is not statistically significant; all other variables included are statistically significant at 95-percent confidence level.

Table 49. Statistical output of NB model estimation for fatal and injury after matching (GD).

Variables	Coef.	S.E.
Constant	-7.642	1.253
Natural logarithm of AADT	0.873	0.145
Presence of guiderail with delineators (1 if present; 0 otherwise)*	-0.021	0.164
Degree of curvature	0.098	0.022
Presence of speed difference above 0 (1 if present; 0 otherwise)	0.735	0.214
Natural logarithm of length (offset)	1.000	n/a

Note: n/a is not applicable; * represents the treatment variable is not statistically significant; all other variables included are significant at 95-percent confidence level.

Table 50. Statistical output of NB model estimation for ROR crash after matching (GD).

Variables	Coef.	S.E.
Constant	-3.616	1.097
Natural logarithm of AADT	0.396	0.129
Presence of guiderail with delineators (1 if present; 0 otherwise)*	0.058	0.158
Degree of curvature	0.111	0.020
Presence of speed difference above 0 (1 if present; 0 otherwise)	0.596	0.205
Natural logarithm of length (offset)	1.000	n/a

Note: n/a is not applicable; * represents the treatment variable is not statistically significant; all other variables included are significant at 95-percent confidence level.

Table 51. Statistical output of NB model estimation for nighttime crash after matching (GD).

Variables	Coef.	S.E.
Constant	-4.991	1.546
Natural logarithm of AADT	0.522	0.180
Presence of guiderail with delineators (1 if present; 0 otherwise)*	-0.157	0.218
Degree of curvature	0.070	0.033
Presence of speed difference above 0 (1 if present; 0 otherwise)	--	--
Natural logarithm of length (offset)	1.000	n/a

Note: n/a means not applicable; — means the variable is not significant thus excluded from the model; * represents the treatment variable is not statistically significant; all other variables included are significant at 95-percent confidence level.

Table 52. CMF estimates of guiderail with delineators (GD).

Guiderail with Delineator	Crash Type	Estimate	95-percent CI	95-percent CI
CMF	Total crash	0.971	0.761	1.239
CMF	Fatal injury	0.979	0.711	1.350
CMF	ROR crash	1.059	0.776	1.445
CMF	Nighttime crash	0.855	0.558	1.309

Safety Effect of Guiderail (G only)

A standard NB regression model was estimated for all the crash types (see table 53, note: n/a is not applicable; * represents the treatment variable which is statistically significant; all other variables included are significant at 95-percent confidence level.

table 54, table 55, and table 56) using the before matching data. The variables included in the models show the association between expected total crashes and the covariates in the model. Negative coefficients of the variables indicate a decrease in crash frequency when those features are present along horizontal curves. The model estimates indicate that the presence of guiderail in curves is expected to reduce all crash types. However, those estimates are produced from the unbalanced raw data, and may be influenced by selection bias.

Table 53. Statistical output of the cross-sectional NB regression for total crashes (G only).

Variable	Coef.	S.E.
Constant	-5.789	0.544
Natural logarithm of AADT	0.708	0.061
Presence of guiderail (1 if present; 0 otherwise)*	-0.310	0.116
Degree of curvature	0.089	0.009
Presence of curve shoulder width under 4 ft (1 if present; 0 otherwise)	0.227	0.088
Presence of speed difference above 0 (1 if present; 0 otherwise)	0.223	0.117
Presence of intersection (1 if present; 0 otherwise)	0.749	0.073
Indicator of Pennsylvania (1 if Pennsylvania; 0 Indiana)	-0.434	0.079
Natural logarithm of length (offset)	1.000	n/a

Note: n/a is not applicable; * represents the treatment variable which is statistically significant; all other variables included are significant at 95-percent confidence level.

Table 54. Statistical output of the cross-sectional NB regression for fatal and injury (G only).

Variables	Coef.	S.E.
Constant	-5.870	0.779
Natural logarithm of AADT	0.560	0.087
Presence of guiderail (1 if present; 0 otherwise)*	-0.240	0.163
Degree of curvature	0.084	0.009
Presence of curve shoulder width under 4 ft (1 if present; 0 otherwise)	0.362	0.123
Presence of speed difference above 0 (1 if present; 0 otherwise)	--	--
Presence of intersection (1 if present; 0 otherwise)	0.612	0.103
Indicator of Pennsylvania (1 if Pennsylvania; 0 Indiana)	0.201	0.103
Natural logarithm of length (offset)	1.000	n/a

Note: n/a not applicable; -- means the variable is not significant thus excluded from the model; * represents the treatment variable and is not statistically significant; all other variables included are significant at 95-percent confidence level;

Table 55. Statistical output of the cross-sectional NB regression for ROR crash (G only).

Variables	Coef.	S.E.
Constant	-4.971	0.638
Natural logarithm of AADT	0.528	0.072
Presence of guiderail (1 if present; 0 otherwise)*	-0.128	0.130
Degree of curvature	0.096	0.009
Presence of curve shoulder width under 4 ft (1 if present; 0 otherwise)	0.410	0.104
Presence of speed difference above 0 (1 if present; 0 otherwise)	0.329	0.126
Presence of intersection (1 if present; 0 otherwise)	0.738	0.084
Indicator of Pennsylvania (1 if Pennsylvania; 0 Indiana)	-0.435	0.091
Natural logarithm of length (offset)	1.000	n/a

Note: n/a is not applicable; * represents the treatment variable and is not statistically significant; all other variables included are significant at 95-percent confidence level.

Table 56. Statistical output of the cross-sectional NB regression for nighttime crash (G only).

Variables	Coef.	S.E.
Constant	-5.279	0.719
Natural logarithm of AADT	0.576	0.083
Presence of guiderail (1 if present; 0 otherwise)*	-0.166	0.147
Degree of curvature	0.076	0.011
Presence of curve shoulder width under 4 ft (1 if present; 0 otherwise)	--	--
Presence of speed difference above 0 (1 if present; 0 otherwise)	0.344	0.157
Presence of intersection (1 if present; 0 otherwise)	0.607	0.098
Indicator of Pennsylvania (1 if Pennsylvania; 0 Indiana)	-0.555	0.110
Natural logarithm of length (offset)	1.000	n/a

Note: n/a not applicable; -- means the variable is not significant thus excluded from the model; * represents the treatment variable and is not statistically significant; all other variables included are significant at 95-percent confidence level.

Propensity Score Matching

Table 57 shows the configuration of propensity score matching. The matching distance is defined by the propensity score, which is the probability of the curves receiving the treatment (G only). In order to yield quality matches, 1:1 NN matching is applied with a caliper width of 0.2. Considering the relatively small sample size in the treatment and reference group, matching with replacement is applied. The statistical output of estimated propensity score model is shown in table 58. The covariates included in the model are the variables related to both the crash outcome and treatment assignment.

Table 57. Propensity score matching configuration (G only).

Matching configuration	Matching configuration
Distance	Propensity score
Algorithm	Nearest neighbour matching
Ratio	1:1
Replacement	Without replacement
Caliper width	0.2

Table 58. Statistical output of propensity score model estimation (G only).

Variable	Coef.	S.E.	z-statistic	p-value
Constant	-0.434	0.783	-0.560	0.579
Natural logarithm of AADT	0.064	0.090	0.720	0.474
Degree of curvature	0.080	0.013	6.290	<0.001
Curve shoulder width under 4 ft (1 if present; 0 otherwise)	-1.485	0.115	-12.890	<0.001
Presence of chevron signs or curve arrow (1 if present; 0 otherwise)	0.076	0.288	0.260	0.792
Presence of speed difference above 0 (1 if present; 0 otherwise)	0.582	0.171	3.400	0.001
Presence of intersection in the curve (1 if present; 0 otherwise)	-0.028	0.112	-0.250	0.802
Indicator of Pennsylvania (1 if Pennsylvania; 0 Indiana)	0.601	0.114	5.290	<0.001
Natural logarithm of curve length	1.0000	Offset	n/a	n/a

Note: Wald chi-square(7) = 260.35; Log-likelihood at convergence = -1216.4987; n/a not applicable.

There were 425 out of 435 observations in the treatment group that were matched with 425 observations in the control group. A total of 10 treatment observations were unmatched and discarded since there were no observations within the caliper width in the reference group. Table 59 and table 60 show the summary statistics before and after matching in each group. Figure 41 and figure 42 show the distribution of the propensity scores in the treatment and reference groups before and after matching. The distribution of propensity scores before matching indicates significant differences between the treatment and reference groups. Most of the observations in the reference group have propensity scores between 0 and 0.1, indicating low probabilities to receive the treatment. On the other hand, the propensity scores in the treatment group are distributed between 0 and 0.4. Although the shapes of the distributions are quite different, there is enough common support to match the treatment and reference groups.

Table 59. Summary of variables before matching (G only).

Variables	Treatment Mean	Treatment S.D.	Reference Mean	Reference S.D.
Propensity score	0.189	0.109	0.113	0.086
ln(AADT)	8.298	0.597	8.173	0.684
ln(length)	-1.737	0.493	-1.801	0.520
Degree of curvature	4.098	4.049	3.827	4.836
Curve shoulder width larger than 4 ft	0.483	0.500	0.818	0.386
Presence of chevron signs or curve arrows	0.069	0.254	0.061	0.239
Speed difference larger than 0	0.149	0.357	0.134	0.341
Presence of intersection	0.529	0.500	0.497	0.500
Indicator of Pennsylvania	0.471	0.500	0.395	0.489

Table 60. Summary of variables after matching (G only).

Variables	Treatment Mean	Treatment S.D.	Reference Mean	Reference S.D.
Propensity score	0.184	0.104	0.182	0.106
ln(AADT)	8.283	0.594	8.286	0.667
ln(length)	-1.750	0.477	-1.749	0.527
Degree of curvature	4.151	4.076	4.004	6.419
Curve shoulder width larger than 4 ft	0.494	0.501	0.504	0.501
Presence of chevron signs or curve arrows	0.071	0.256	0.068	0.252
Speed difference larger than 0	0.137	0.344	0.139	0.346
Presence of intersection	0.522	0.500	0.508	0.501
Indicator of Pennsylvania	0.459	0.499	0.464	0.499

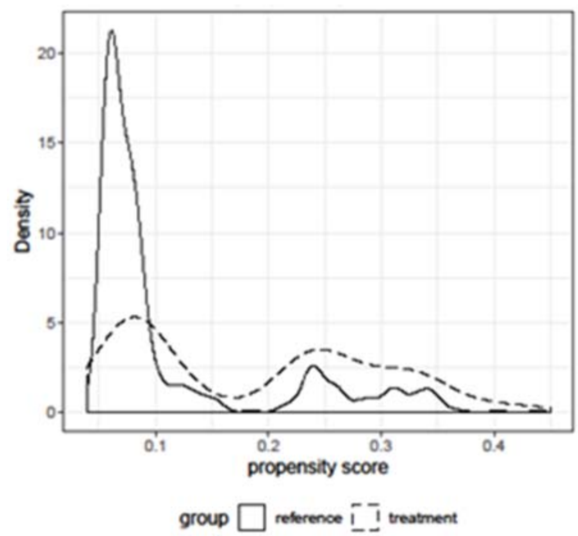

Figure 41. Graph. Propensity score distributions before matching (G only).

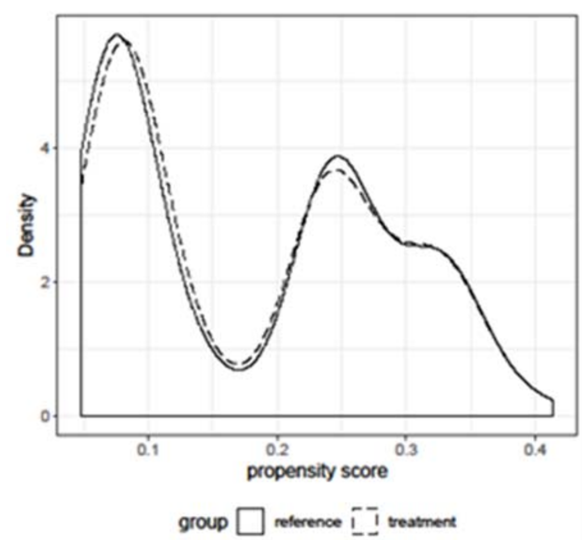

Figure 42. Graph. Propensity score distributions after matching (G only).

Covariate Balance Assessment

Table 61 and table 62 show the mean and absolute standardized mean difference ASMD of the propensity score and each covariate before and after matching. An ASMD closer to zero indicates better balance of the covariates between the treatment and reference groups. An ASMD larger than 0.20 indicates a statistically significant difference between the treatment and reference groups. As shown in table 61, the ASMD of the propensity score is reduced by 98.4 percent, from 0.778 to 0.012. The average ASMD is reduced from 0.176 to 0.014 as a result of matching. The last column in table 62 shows that matching significantly improved the balance among all covariates. The ASMD comparison of each covariate before and after matching is shown in figure 43. Compared to the before-matching data, all of the ASMD values are close to zero after matching, indicating that the covariates are similar in the treatment and reference groups.

Table 61. Mean and ASMD difference before and after matching (G only).

Variable	Mean Difference Before	Mean Difference After	ASMD Before	ASMD After	Percentage of Balance Improved
Propensity score	0.077	0.001	0.778	0.012	98.4%

Table 62. Mean and ASMD covariates before and after matching (G only).

Covariates	Mean Difference Before	Mean Difference After	ASMD Before	ASMD After	Percentage of Balance Improved
ln(AADT)	0.125	-0.003	0.194	0.005	97.4%
ln(length)	0.063	-0.001	0.125	0.002	98.7%
Degree of curvature	0.271	0.146	0.061	0.027	46.0%
Curve shoulder width larger than 4 ft	-0.335	-0.009	0.750	0.019	97.2%
Presence of chevron signs or curve arrows	0.008	0.002	0.034	0.009	71.5%
Speed difference larger than 0	0.015	-0.002	0.044	0.007	84.6%
Presence of intersection	0.032	0.014	0.064	0.028	55.8%
Indicator of Pennsylvania	0.0767	-0.0047	0.155	0.009	93.9%
Mean	n/a	n/a	0.176	0.014	n/a

Note: n/a not applicable.

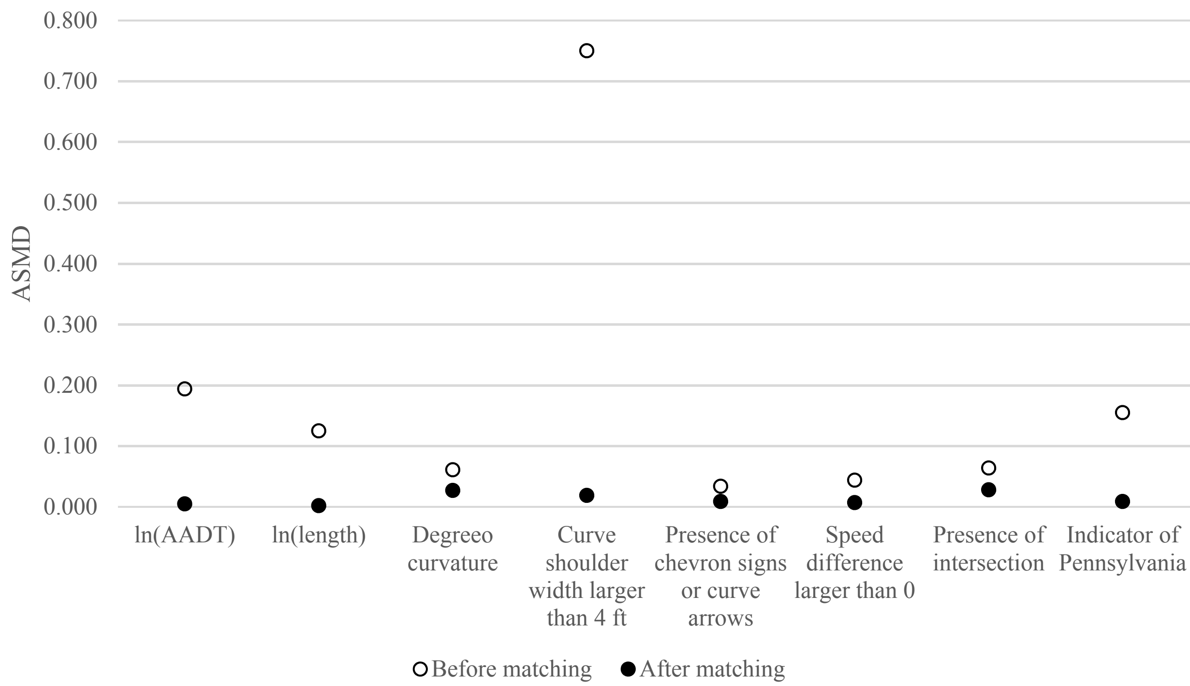

Figure 43. Graph. ASMD before and after matching (G only).

CMF Estimates

Table 63, table 64, table 65, table 66, and table 67 show a NB regression model after matching, as well as the CMF estimates of each crash type. The CMF point estimates are all less than 1.0 for total crashes and fatal and injury crashes, indicating that guiderail offers a safety benefit for these crash types along horizontal curves on two-lane roadways. For the ROR-related and night-time crash frequency, the presence of only guiderail along horizontal curves has a CMF point estimate great than 1.0. However, all 95-percent confidence intervals for the CMFs contain 1.0, suggesting that additional research should be completed to compile a larger sample of study sites.

Table 63. Statistical output of NB model estimation for total crashes after matching (G only).

Variable	Coef.	S.E.
Constant	-4.551	1.127
Natural logarithm of AADT	0.539	0.129
Presence of guiderail with delineators (1 if present; 0 otherwise)*	-0.168	0.144
Degree of curvature	0.097	0.015
Presence of curve shoulder width under 4 ft (1 if present; 0 otherwise)	--	--
Presence of intersection (1 if present; 0 otherwise)	0.703	0.149
Indicator of Pennsylvania (1 if Pennsylvania; 0 Indiana)	--	--
Natural logarithm of length (offset)	1.000	n/a

Note: n/a not applicable; -- means the variable is not significant thus excluded from the model; *represents the treatment variable and is not statistically significant; all other variables included are significant at 95-percent confidence level.

Table 64. Statistical output of NB model estimation for fatal and injury after matching (G only).

Variable	Coef.	S.E.
Constant	-2.891	1.572
Natural logarithm of AADT	0.226	0.179
Presence of guiderail with delineators (1 if present; 0 otherwise)*	-0.324	0.205
Degree of curvature	0.090	0.018
Presence of curve shoulder width under 4 ft (1 if present; 0 otherwise)	--	--
Presence of intersection (1 if present; 0 otherwise)	0.398	0.221
Indicator of Pennsylvania (1 if Pennsylvania; 0 Indiana)	0.398	0.221
Natural logarithm of length (offset)	1.000	n/a

Note: n/a not applicable; -- means the variable is not significant thus excluded from the model; *represents the treatment variable and is not statistically significant; all other variables included are significant at 95-percent confidence level.

Table 65. Statistical output of NB model estimation for ROR crash after matching (G only).

Variable	Coef.	S.E.
Constant	-4.482	1.400
Natural logarithm of AADT	0.445	0.159
Presence of guiderail with delineators (1 if present; 0 otherwise)*	0.088	0.171
Degree of curvature	0.096	0.016
Presence of curve shoulder width under 4 ft (1 if present; 0 otherwise)	0.400	0.183
Presence of intersection (1 if present; 0 otherwise)	0.833	0.185
Indicator of Pennsylvania (1 if Pennsylvania; 0 Indiana)	-0.459	0.187
Natural logarithm of length (offset)	1.000	n/a

Note: n/a not applicable; *represents the treatment variable and is not statistically significant; all other variables included are significant at 95-percent confidence level.

Table 66. Statistical output of NB model estimation for nighttime crash after matching (G only).

Variable	Coef.	S.E.
Constant	-6.158	1.612
Natural logarithm of AADT	0.625	0.181
Presence of guiderail with delineators (1 if present; 0 otherwise)*	0.026	0.196
Degree of curvature	0.065	0.019
Presence of curve shoulder width under 4 ft (1 if present; 0 otherwise)	0.394	0.204
Presence of intersection (1 if present; 0 otherwise)	0.882	0.215
Indicator of Pennsylvania (1 if Pennsylvania; 0 Indiana)	-0.460	0.212
Natural logarithm of length (offset)	1.000	n/a

Note: n/a not applicable; *represents the treatment variable and is not statistically significant; all other variables included are significant at 95-percent confidence level.

Table 67. CMF estimates of guiderail with delineators (G only).

Guiderail	Crash Type	Estimate	95-percent CI	95-percent CI
CMF	Total crash	0.845	0.638	1.119
CMF	Fatal injury	0.723	0.484	1.080
CMF	ROR crash	1.092	0.781	1.528
CMF	Nighttime crash	1.026	0.699	1.507

Disaggregate Analysis

The objective of the disaggregate analysis was to verify if the safety effect estimates are consistent under differing application scenarios. In this study, the degree of curve was disaggregated into separate categories to determine if the CMFs for guiderail with delineators or guiderail only differed based on curve sharpness. The data were stratified at a degree of curvature threshold equal to four. The CMFs were estimated using the propensity score matching method for each degree of curvature category.

Guiderail with Delineators

Base on the disaggregation, there were 101 sites (505 observations) included in the category when the degree of curve is greater than 4, including 37 treatment and 64 reference sites. There were 247 sites (1235 observations) assigned to the category with a degree of curve less than 4, including 64 treatment and 183 reference sites. Table 68 and table 69 show the summary statistics of each crash type and traffic volume for the disaggregate data.

Table 68. Summary mean for each degree of curvature category (GD).

Category	Observation	Total Crash	Fatal and Injury	ROR Crash	Nighttime Crash	AADT
Degree of curvature ≥ 4	505	0.287	0.154	0.248	0.113	2757
Degree of curvature < 4	1235	0.301	0.158	0.167	0.100	4159

Table 69. Summary standard deviation for each degree of curvature category (GD).

Category	Observation	Total Crash	Fatal and Injury	ROR Crash	Nighttime Crash	AADT
Degree of curvature ≥ 4	505	0.583	0.408	0.538	0.341	2113
Degree of curvature < 4	1235	0.611	0.413	0.431	0.326	2416

Table 70 and table 71 show the disaggregate CMF estimates for each crash type. With the exception of the ROR crash type in the degree of curve greater than 4 category, all CMF point estimates are lower than 1.0. For sharper curves, with a degree of curvature equal to or greater than 4, the CMFs for each crash type (excluding total crashes) is lower relative to the flatter curve category. This suggests that guiderail with delineators may offer greater safety benefits on sharp horizontal curves relative to flat horizontal curves. However, the 95-percent confidence intervals include 1.0 for all CMFs, so additional research is recommended to increase the sample size of the guiderail with delineators treatment.

Table 70. Disaggregate analysis for CMF estimates using a degree of curvature greater than or equal to four, for crash type using propensity score matching (GD).

Category	Total Crash	Fatal Injury	ROR Crash	Nighttime Crash
CMF	0.976	0.871	0.845	0.622
95-percent confidence interval	0.576	0.423	0.475	0.257
95-percent confidence interval	1.654	1.791	1.503	1.500

Table 71. Disaggregate analysis for CMF estimates using a degree of curvature less than four, for crash type using propensity score matching (GD).

Category	Total Crash	Fatal Injury	ROR Crash	Nighttime Crash
CMF	0.931	0.933	1.173	0.706
95-percent confidence interval	0.702	0.646	0.804	0.435
95-percent confidence interval	1.234	1.348	1.712	1.146

Guiderail

There were 192 sites (960 observations) assigned to the category of degree of curvature larger than 4, including 26 treatment sites and 166 reference sites. There were 521 sites (2,605 observations) assigned to the degree of curvature smaller than 4 category, including 61 treatment sites and 460 reference sites. Table 72 and table 73 show the summary statistics of each crash type and traffic volume for the disaggregate data.

Table 72. Summary mean for each degree of curvature category (G only).

Category	Observations	Total Crash	Fatal Injury	ROR Crash	Nighttime Crash	AADT
Degree of curvature ≥ 4	960	0.443	0.152	0.325	0.191	3273
Degree of curvature < 4	2605	0.477	0.148	0.264	0.185	4804

Table 73. Summary standard deviation for each degree of curvature category (G only).

Category	Observations	Total Crash	Fatal Injury	ROR Crash	Nighttime Crash	AADT
Degree of curvature ≥ 4	960	0.966	0.426	0.747	0.544	2114
Degree of curvature < 4	2605	1.368	0.434	0.669	0.540	2850

Table 74 and table 75 show the disaggregate CMF point estimates for each crash type. All CMFs for flatter curves (degree of curve smaller than 4) were lower than the CMFs for the sharper curves. This suggests that installing guiderail on flat the horizontal curves on two-lane rural highways may offer greater safety benefits relative to installing guiderail on sharper horizontal curves. The CMFs for ROR and nighttime crashes on sharp horizontal curves have point estimates that exceed 1.0.

Table 74. Disaggregate analysis for CMF estimates degree of curvature greater than or equal to four, for crash types using propensity score matching (G only).

Category	Total Crash	Fatal Injury	ROR Crash	Nighttime Crash
CMF	0.954	0.943	1.159	1.082
95-percent confidence interval	0.576	0.493	0.665	0.564
95-percent confidence interval	1.581	1.802	2.021	2.074

Table 75. Disaggregate analysis for CMF estimates for degree of curvature less than four, for crash types using propensity score matching (G only).

Category	Total Crash	Fatal Injury	ROR Crash	Nighttime Crash
CMF	0.805	0.647	0.868	1.001
95-percent confidence interval	0.551	0.377	0.536	0.584
95-percent confidence interval	1.178	1.108	1.406	1.714

Summary of Results

Table 76 shows the expected CMF effect sizes for each crash type based on the literature review and engineering intuition. An up-arrow indicates a negative safety effect, or a CMF larger than 1.0, while a down-arrow indicates a positive safety effect, or a CMF smaller than 1.0. A two-sided horizontal arrow indicates both safety benefits and dis-benefits are possible. As shown in table 32, table 33, table 34, table 35, table 36, and table 37, guiderail with delineators, and delineators only were expected to increase total crash frequency but reduce the fatal and injury crash frequency. According to the literature review, the presence of guiderail has been shown to both positively and negatively affect total crash frequency but has positive safety effects on fatal and injury crashes. For ROR crashes, the presence of guiderail (whether it is with delineators or not) is expected to increase the expected crash frequency, which was also indicated in the literature. The presence of delineators was expected to be an effective safety countermeasure with regards to nighttime crashes, which has also been verified in the literature. However, the presence of guiderail only is expected to increase nighttime crash frequency.

Table 76. Expected CMFs of the countermeasures for each crash type.

Treatment	Total Crash	Fatal Injury	ROR Crash	Dark Crash
Guiderail with delineator	↑	↓	↑	↓
Guiderail	↔	↓	↑	↑
Delineator	↑	↓	↓	↓

Table 77. Summary of CMF estimates for all sites of guiderail with delineator, guiderail only, and delineators.

Safety Effect	Total Crash	Fatal Injury	ROR Crash	Nighttime Crash
CMF_{gd}	0.971	0.979	1.059	0.855
CMF_g	0.845	0.723	1.092	1.026
CMF_d*	1.149	1.354	0.970	0.833

Note: * the safety effect of delineators (CMF_d) is estimated by taking the ratio of CMF_{gd} and CMF_g.

Table 78. Summary of CMF estimates for degree of curvature greater than or equal to four of guiderail with delineator, guiderail only, and delineators.

Safety Effect	Total Crash	Fatal Injury	ROR Crash	Nighttime Crash
CMF_{gd}	0.976	0.871	0.845	0.622
CMF_g	0.954	0.943	1.159	1.082
CMF_d*	1.023	0.924	0.729	0.575

Note: * the safety effect of delineators (CMF_d) is estimated by taking the ratio of CMF_{gd} and CMF_g.

Table 79. Summary of CMF estimates for degree of curvature less than four of guiderail with delineator, guiderail only, and delineators.

Safety Effect	Total Crash	Fatal Injury	ROR Crash	Nighttime Crash
CMF_{gd}	0.931	0.933	1.173	0.706
CMF_g	0.805	0.647	0.868	1.001
CMF_d*	1.156	1.443	1.351	0.706

Note: * the safety effect of delineators (CMF_d) is estimated by taking the ratio of CMF_{gd} and CMF_g.

As shown in table 77, table 78, and table 79 the CMF point estimates from the present study were often consistent with expectations. The CMF estimates of guiderail with delineators (CMF_{gd}), and guiderail only (CMF_g) were used to derive CMF estimates for delineators only (CMF_d). Regarding the safety effect of the only presence of guiderail, the results show that it has safety benefits for total crashes, as well as fatal and injury crashes. The estimates are consistent with expectations. However, for ROR and nighttime crashes, is the CMFs for guiderail exceed

1.0 in all conditions, except for the ROR crash type in mild curves. These results are also consistent with engineering expectations.

The CMF for delineators (CMF_d) are inferred from the ratio of CMF_{gd} relative to CMF_g. The results indicate that all the CMF_d for total crashes are larger than 1.0, which is consistent with engineering expectation. However, for sharp curves, the CMF for delineators is closer to 1.0 when compared to the CMF for mild curves. Regarding fatal and injury crashes, the CMF for delineators is less than 1.0 for sharp curves, but greater than 1.0 for mild curves. Similar findings resulted for ROR crashes. For nighttime crashes, the CMF for delineators was less than 1.0 for both sharp and flat horizontal curves. The CMFs were generally consistent with engineering expectations for sharper horizontal curves.

DISCUSSION AND CONCLUSIONS

This study used the RID to evaluate the safety effects of guiderail only, guiderail with delineators, and delineators along horizontal curves on two-lane rural highways in Pennsylvania. A propensity scores-potential outcomes framework was used to compare segments of roadway with the treatment of interest to similar roadways without the treatment. The results show that guiderail with delineators have safety benefits for total crashes, fatal and injury crashes, ROR, and nighttime crashes on horizontal curves that are four degrees or sharper. On flatter horizontal curves, the safety effects are also positive for total, fatal and injury, and nighttime crashes, but the present dataset does not show a ROR crash safety benefit for guiderail with delineators on horizontal curves that are flatter than four degrees.

For guiderail only, safety benefits were found for total and fatal and injury crashes on sharp horizontal curves, while the CMFs exceeded 1.0 for ROR and nighttime crashes on sharp curves. Adding guiderail to the roadside introduces a fixed object, so an increase in ROR crashes on sharp horizontal curves is consistent with expectations. Similarly, at night, guiderail without delineators will have limited visibility, so it was expected that expected crash frequencies at night may increase when guiderail is present along horizontal curves. Based on the present study, the presence of guiderail appears to offer total crash, fatal and injury crash, and ROR crash benefits on flat horizontal curves. This may be the result of the decreased likelihood that vehicles will leave the roadway on flat horizontal curves.

The CMFs for delineators were derived from the guiderail with delineators and guiderail only CMFs. On curves sharper than four degrees, delineators offered a safety benefit relative to fatal and injury, ROR, and nighttime crashes, presumably because the added guidance to drivers is helping to define sharp curves and subsequently lowering travel speeds and reducing severe crash types at night. On flatter horizontal curves, the CMFs for delineators exceed 1.0 for total, fatal and injury, and ROR crashes. The delineators may be promoting higher speeds on these curves because the delineators make the curve appear flat, thus promoting high-speed roadway departure events.

The data in this study were acquired from Indiana and Pennsylvania. It is recommended that additional two-lane rural highway curves be identified in other States to confirm the results presented herein. In particular, this study was unable to consider guiderail or delineator effectiveness in States with significantly different geographic or climatic conditions. Further, information related to the offset of the roadside barrier was not available in the RID data, and the offset may affect the frequency and severity of crashes on horizontal curves.

CHAPTER 7. HORIZONTAL CURVE WARNING PAVEMENT MARKINGS

Horizontal curve warning pavement markings are a safety treatment that includes an arrow in the direction of the curve with the word “slow” under the arrow, painted on the road located near the beginning of the horizontal curve in the direction of travel. An alternative version of this treatment includes the curve direction arrow or a speed advisory in place of the word “slow.” Figure 44 shows the curve-slow version of the treatment. The objective of this treatment is to warn drivers of an upcoming, sharp horizontal curve. The intended purpose of the pavement marking is to reduce operating speeds when entering a curve, thereby reducing roadway departure crashes. No previous studies have investigated the safety effects of this treatment.

The objective of this evaluation is to analyze the safety effectiveness of the horizontal curve warning pavement marking and to develop crash modification factors (CMFs) for its use.

Figure 44. Photo. Horizontal curve warning pavement marking (Retting and Farmer, 1998).

BACKGROUND

Published research on the horizontal curve warning pavement marking has focused on operational evaluations. A speed evaluation of this treatment by Retting and Farmer (1998) involved an arrow curved to the left with the word “slow” painted beneath it at a location approximately 220 ft before the point of curvature (PC) of a left-hand horizontal curve. A single curve on a two-lane suburban highway in northern Virginia with a paved roadway width of 20 ft received the treatment. A separate curve on the same highway was used as a control site; only this curve did not contain the horizontal curve pavement marking warning. The posted speed limit was 35 mph, while an advisory speed of 15 mph was posted approximately 500 ft before the curve with the treatment.

Vehicle operating speeds were collected at locations 90-ft prior to the PC as well as at a location 650 ft prior to the PC of the treated curve. Speeds at the control curve were collected 100 ft prior to the PC. Operating speed data were collected five months prior to installation of the pavement marking and two weeks after installation. Logistic regression was used to model the effect of the pavement marking on vehicle operating speeds. The results indicated a statistically significant decrease in mean operating speeds at the location before the PC, based on a comparison of operating speeds at the treatment and the control sites, before and after the pavement marking sign was deployed. The magnitude of the speed reduction was 7 percent (2.4 mph) after the horizontal curve warning pavement marking was implemented.

The on-pavement curve warning pavement markings were also evaluated by Hallmark et al. (2012) This study included markings at two horizontal curve locations on two-lane rural highways in Iowa. No comparison locations were used in the evaluation. Operating speed data at each of the curves were collected one month before installation, one month after installation, and 12 months after installation of the treatments. Vehicle speeds were collected at the PC, midpoint of the horizontal curve, and point of tangency (PT) at each curve location. Changes in mean speed for each of the locations were then analyzed using t-tests. The percentage of vehicles exceeding the posted speed limit was also analyzed using a test of proportions.

When aggregated, the results indicated that the curve warning pavement markings were not effective in reducing either the mean or 85th-percentile operating speeds in the short- or long-term after periods at both curve locations. When considering each curve separately, the results indicated increased speeds at one curve and decreased speeds at the second curve location.

In summary, the safety effects of horizontal curve pavement marking warning signs have not be evaluated. The operational effects, based solely on operating speed measures, are not clear. Research has found that the horizontal curve pavement markings reduce operating speeds when approaching the curve, while other research has found that operating speed within the limits of the curve are not affected by the markings.

ANALYSIS METHOD

An observational before-after safety study is considered a robust evaluation method for assessing the effectiveness of safety countermeasures. This method requires that data be available from both before and after the implementation of the countermeasure. An overview of the empirical Bayes (EB) method is provided below: (Hauer 1997, Persaud and Lyon 2007, Park et al. 2012)

- *Step 1*: Predict what the safety performance would have been in the after period had the countermeasure not been implemented.
- *Step 2*: Estimate what the actual safety performance was in the after period with the countermeasure.
- *Step 3*: Compare the results of *Step 1* and *Step 2*.

In *Step 1*, a reference group is used to account for the effects of traffic volume and temporal effects on safety due to the variation in weather, demographics, and crash reporting. This is done by estimating a safety performance function (SPF) relating crash frequency to traffic flow and other safety-influencing features present at the study sites. The development of the SPF in this evaluation is based on a reference group of horizontal curve sites which were not treated with any safety countermeasures during the analysis period.

Variables considered for SPF estimation included crash, traffic volume, and roadway and roadside features. Standard negative binomial (NB) regression methods were used to estimate the SPFs using the STATA statistical software.

The method described below is consistent with Persaud and Lyon (2007). In the EB method, the expected number of crashes had no treatment been applied (for the after period – $N_{EB,After}$) is based on the SPF, which is first used to estimate the number of crashes expected in each year of the before period at locations with traffic volumes and other site characteristics similar to the treatment site being analyzed. The sum of the annual SPF estimates $(N_{pred,before})$ are combined with the count of crashes $(N_{Observed,before})$ in the before period at the treatment site to obtain an estimate of the expected number of crashes $(N_{EB,before})$ before the treatment was applied. This estimate of $N_{EB,before}$ was developed as seen in figure 45.

$$N_{EB,before} = wN_{pred,before} + (1-w)N_{Obs,before}$$

Figure 45. Equation. Expected number of crashes in the before period EB analysis.

Given that the SPF was specified using the standard negative binomial, the weight *w* is estimated as follows in figure 46.

$$w = \frac{1}{(1+\alpha N_{pred,before})}$$

Figure 46. Equation. EB weighting factor.

A factor is then applied to $N_{EB,before}$ to account for the length of the after period and differences in traffic volumes between the before and after periods. This factor is the sum of the annual SPF predictions for the after period divided by $N_{pred,before}$, the sum of the predictions in the before period. After applying this factor, an estimate of $N_{EB,After}$ results. Additionally, an estimate of the variance of the expected number of crashes that would have occurred in the after period without the treatment was computed following the method described by Persaud and Lyon (2007).

The estimate of $N_{EB,After}$ is summed over all sites in a treatment group of interest $\left(\sum N_{EB,after}\right)$ and compared with the count of crashes during the after period in that group $\left(\sum N_{Obs,after}\right)$. The variance of $N_{EB,After}$ is also summed over all sections in the group of interest (i.e., $\sum VAR(N_{EB,after})$).

The index of safety effectiveness *(θ)* is determined using the following in figure 47.

$$\theta = \frac{\sum N_{Obs,after} \Big/ \sum N_{EB,after}}{1 + \sum VAR\left(N_{EB,after}\right) \Big/ \left(\sum N_{EB,after}\right)^2}$$

Figure 47. Equation. Index of effectiveness.

The standard deviation of θ is determined using the following in figure 48.

$$stdev(\theta) = \sqrt{\theta^2 \left(\frac{\frac{1}{\sum N_{Obs,after}} + \frac{\sum VAR\left(N_{EB,after}\right)}{\left(\sum N_{EB,after}\right)^2}}{1 + \sum VAR\left(N_{EB,after}\right) \Big/ \left(\sum N_{EB,after}\right)^2} \right)}$$

Figure 48. Equation. Standard deviation of index of effectiveness.

The 95-percent confidence interval for θ was found by adding and subtracting 1.96 times $stdev(\theta)$ from θ.

SPF Estimation

The general functional form of the SPF to be estimated is shown below in figure 49 and is the same for all crash types considered (e.g., SVROR or total crashes). The expected crash frequency (per mi per year) was the dependent variable and the traffic volume (AADT), and other site-characteristic data ($X_1, \ldots, X_n$) were entered into the model as predictor (independent) variables. Only data from the reference group were used to estimate the SPFs (i.e., horizontal curves without the treatment). The segment length variable (L) is the horizontal curve length. The β_0 term is the regression constant, while the remaining β (β_{Length}, β_{AADT}, β_1, …, β_n) were estimated from the data sample using NB regression.

$$Crashes/mile/year = L^{\beta_{Length}} \times AADT^{\beta_{AADT}} \times \exp(\beta_0 + \beta_1 X_1 + \ldots + \beta_n X_n)$$

Figure 49. Equation. SPF.

DATA

Data for this project were collected and compiled using several existing databases maintained by Penn State and the Pennsylvania Department of Transportation (PennDOT). The roadway inventory database (RID) was developed at Penn State using PennDOT roadway inventory data, on-line video photologs, and Google Earth, and included all two-lane rural highways owned and operated by PennDOT. It should be noted that PennDOT codifies the State roadway network using four digits (e.g., State route 2024), but this study only considers roadways with three or fewer digits (e.g., State route 0823, 0016, or 0006). This is because traffic volume data on the four-digit State routes are often missing because these roadways are often low-speed, low-volume roadways that traverse short lengths. The online video photologs are maintained by PennDOT and were used to collect roadside hazard rating (based on the 1 to 7 scale proposed by Zegeer et al., 1987), the presence of centerline and shoulder rumble strips, and other pertinent roadway and roadside features. Google Earth was used to collect curve radius and length data. These variables were combined with existing variables in the PennDOT roadway inventory data, implementation dates and locations of the on-pavement curve markings, and crash data files to create the database used for this evaluation. Crash data were available for the years 2005-2013 (inclusive).

Horizontal curves with the on-pavement curve warning pavement markings in Pennsylvania two-lane highways with installation dates between 2000 and 2015 were obtained from PennDOT. However, since crash data were only available for the years 2005-2013, only horizontal curves with treatment installations between the years 2006 and 2012 could be used in the analysis (i.e., that have at least one year of crash data in both the before period (i.e., treated in 2006) or at least one year in the after period (i.e., treated in 2012)). Thus, in the analysis, a total of 263 treated horizontal curves were included. This included 93 curves with installation dates in 2006, 23 in 2007, 4 in 2008, 16 in 2009, 42 in 2010, 41 in 2011, and 44 in 2012. These curves were used as the treatment group. Other horizontal curves with installation dates prior to 2006 or in 2013 were removed from the database prior to analysis. The reference group included 21,902 horizontal curves on two-lane highways in Pennsylvania that did not receive the horizontal curve warning pavement marking at any time prior to 2014. These horizontal curves were used to estimate the SPFs.

Variable definitions in the analysis database are provided in table 80. Descriptive statistics for the reference group are provided in table 81 and table 82. Descriptive statistics for the treated curves in the before period are provided in table 83 and table 84. Descriptive statistics for the treated curves in the after period are provided in table 85 and table 86.

The horizontal curves with on-pavement curve warning pavement markings that were treated between 2006 and 2012 each only had 8 years of data in the final analysis database due to removing data for the year the markings were implemented (to reduce the potential bias associated with knowing the specific day when the markings were painted, and associated work zone locations around the treatment sites). All locations in the reference group contained up to 9 years of data. There were a few curves in the reference group that traffic volumes and other data were missing for some years. Due to the large size of the database, the observations with missing values were removed from the analysis files prior to analysis.

RESULTS

The EB method was applied using the 263 treated curves. The target crash types were run-off-road, nighttime, nighttime run-off-road (ROR), and nighttime fatal and injury crashes. Total crashes, as well as total fatal and injury crashes (for both day and night combined), were also evaluated. The estimated SPFs for each of the crash types evaluated are shown in table 87, table 88, table 89, table 90, table 91, and table 92. As shown, the same variables were included in the SPFs for all crash types with the exception of not including roadside hazard rating indicator variables in the SPFs for nighttime, nighttime ROR, and nighttime fatal + injury crashes. The roadside hazard rating variables were not included in these SPFs due to high t-statistics and p-values (e.g., p-values > 0.625) and unexpected signs and magnitudes of the estimated coefficients (i.e., higher ratings were associated with fewer crash frequencies).

Table 80. Variable descriptions.

Variable	Description
AADT	Annual average daily traffic
Length (mi)	Curve length in mi
Shoulder width (ft)	Shoulder width in ft
Degree of curve	The degree of curvature
Total crashes	The total number of crashes
Fatal + injury crashes	The total number of fatal and injury crashes
ROR crashes	The total number of crashes
Nighttime crashes	The total number of crashes
Nighttime ROR crashes	The total number of crashes
Nighttime fatal + injury crashes	The total number of crashes
Posted speed 50-55 mph	If the posted speed is 50-55 mph = 1, 0 = otherwise
Year = 2005	If the year is 2005 = , 0 = otherwise
Year = 2006	If the year is 2006 = , 0 = otherwise
Year = 2007	If the year is 2007 = , 0 = otherwise
Year = 2008	If the year is 2008 = , 0 = otherwise
Year = 2009	If the year is 2009 = , 0 = otherwise
Year = 2010	If the year is 20010 = , 0 = otherwise
Year = 2011	If the year is 2011 = , 0 = otherwise
Year = 2012	If the year is 2012 = , 0 = otherwise
Passing zone present	If a passing zone is present = 1, 0 = otherwise
Centerline rumble strips	If a centerline rumble strips are present = 1, 0 = otherwise
Shoulder rumble strips	If a shoulder rumble strips are present = 1, 0 = otherwise
Roadside hazard rating = 4 or 5	If roadside hazard rating is 4 or 5 = 1, 0 = otherwise
Roadside hazard rating = 6 or 7	If roadside hazard rating is 6 or 7 = 1, 0 = otherwise

Table 81. Descriptive statistics for reference group (21, 902 curves).

Variable	Mean	S.D.	Min	Max
AADT	2597	2395	100	15,998
Length (mi)	0.082	0.066	0	0.567
Shoulder width (ft)	3.571	1.890	0	12
Degree of curve	7.724	8.416	0.852	94.307
Total crashes	0.104	0.389	0	12
Fatal + injury crashes	0.057	0.267	0	7
ROR crashes	0.069	0.293	0	7
Nighttime crashes	0.040	0.214	0	6
Nighttime ROR crashes	0.031	0.185	0	4
Nighttime fatal + injury crashes	0.021	0.150	0	3

Table 82. Descriptive statistics for reference group indicator variables (21, 902 curves).

Variable	Proportion in Sample
Posted speed 50-55 mph	0.393
Year = 2005	0.111
Year = 2006	0.111
Year = 2007	0.111
Year = 2008	0.111
Year = 2009	0.111
Year = 2010	0.111
Year = 2011	0.111
Year = 2012	0.111
Passing zone present	0.164
Centerline rumble strips	0.215
Shoulder rumble strips	0.069
Roadside hazard rating = 4 or 5	0.720
Roadside hazard rating = 6 or 7	0.244

Table 83. Descriptive statistics for treated group in the before period (263 curves).

Variable	Mean	S.D.	Min	Max
AADT	2802	2491	100	15,998
Length (mi)	0.075	0.063	0.010	0.471
Shoulder Width (ft)	3.600	1.972	0	12
Degree of curve	10.017	13.163	0.854	94.558
Total crashes	0.044	0.242	0	3
Fatal + injury crashes	0.019	0.152	0	2
ROR crashes	0.020	0.142	0	1
Nighttime crashes	0.017	0.131	0	1
Nighttime ROR crashes	0.012	0.110	0	1
Nighttime fatal + injury crashes	0.008	0.090	0	1

Table 84. Descriptive statistics for treated group in the before period indicator variables (263 curves).

Variable	Proportion in Sample
Posted Speed 50-55 mph	0.416
Year = 2005	0.269
Year = 2006	0.174
Year = 2007	0.150
Year = 2008	0.146
Year = 2009	0.130
Year = 2010	0.087
Year = 2011	0.045
Year = 2012	0.000
Passing zone present	0.182
Centerline rumble strips	0.205
Shoulder rumble strips	0.079
Roadside hazard rating = 4 or 5	0.739
Roadside hazard rating = 6 or 7	0.186

Table 85. Descriptive statistics for treated group in the after period (263 curves).

Variable	Mean	S.D.	Min	Max
AADT	2584	2258	103	14,493
Length (mi)	0.075	0.065	0.0099	0.471
Shoulder width (ft)	3.666	2.072	0	12
Degree of curve	8.964	9.553	0.854	94.558
Total crashes	0.055	0.240	0	2
Fatal + injury crashes	0.034	0.181	0	1
ROR crashes	0.046	0.218	0	2
Nighttime crashes	0.026	0.159	0	1
Nighttime ROR crashes	0.021	0.145	0	1
Nighttime fatal + injury crashes	0.015	0.122	0	1

Table 86. Descriptive statistics for treated group in the after period indicator variables (263 curves).

Variable	Proportion in Sample
Posted Speed 50-55 mph	0.371
Year = 2005	0.000
Year = 2006	0.000
Year = 2007	0.083
Year = 2008	0.103
Year = 2009	0.107
Year = 2010	0.121
Year = 2011	0.158
Year = 2012	0.195
Passing zone present	0.169
Centerline rumble strips	0.248
Shoulder rumble strips	0.102
Roadside hazard rating = 4 or 5	0.765
Roadside hazard rating = 6 or 7	0.194

Table 87. Estimated SPFs for total crashes.*

Variable	Coef.	S.E.	P-Value
Natural log of AADT	0.762	0.010	<0.001
Natural log of length	0.823	0.013	<0.001
Posted speed 50-55 mph	-0.047	0.018	0.008
Year = 2005	-0.841	0.039	<0.001
Year = 2006	-0.214	0.033	<0.001
Year = 2007	-0.195	0.033	<0.001
Year = 2008	-0.028	0.032	0.375
Year = 2009	-0.058	0.032	0.071
Year = 2010	-0.053	0.032	0.099
Year = 2011	-0.013	0.032	0.684
Year = 2012	-0.449	0.035	<0.001
Degree of curve	0.039	0.001	<0.001
Shoulder width (ft)	-0.058	0.005	<0.001
Passing zone present	-0.085	0.022	<0.001
Centerline rumble strips	0.084	0.020	<0.001
Shoulder rumble strips	-0.024	0.031	0.435
Roadside hazard rating = 4 or 5	0.014	0.044	0.749
Roadside hazard rating = 6 or 7	0.035	0.047	0.452
Constant	-5.981	0.100	<0.001
Overdispersion parameter (α)	1.724	0.048	<0.001

Note: * represents pseudo r-square = 0.0880.

Table 88. Estimated SPFs for fatal and injury crashes.*

Variable	Coef.	S.E.	P-Value
Natural log of AADT	0.727	0.013	<0.001
Natural log of length	0.817	0.016	<0.001
Posted Speed 50-55 mph	-0.040	0.023	0.077
Year = 2005	-0.761	0.050	<0.001
Year = 2006	-0.139	0.042	0.001
Year = 2007	-0.186	0.043	<0.001
Year = 2008	-0.025	0.041	0.539
Year = 2009	-0.023	0.042	0.575
Year = 2010	0.012	0.041	0.773
Year = 2011	-0.056	0.042	0.178
Year = 2012	-0.391	0.045	<0.001
Degree of curve	0.038	0.001	<0.001
Shoulder width (ft)	-0.051	0.006	<0.001
Passing zone present	-0.085	0.028	0.003
Centerline rumble strips	0.094	0.025	<0.001
Shoulder rumble strips	-0.042	0.040	0.287
Roadside hazard rating = 4 or 5	0.003	0.057	0.955
Roadside hazard rating = 6 or 7	0.024	0.060	0.687
Constant	-6.361	0.128	<0.001
Overdispersion Parameter (α)	1.885	0.083	<0.001

Note: * represents pseudo r-square = 0.0785.

Table 89. Estimated SPFs for ROR crashes.*

Variable	Coef.	S.E.	P-Value
Natural log of AADT	0.574	0.012	<0.001
Natural log of length	0.835	0.015	<0.001
Posted speed 50-55 mph	-0.005	0.020	0.801
Year = 2005	-0.959	0.047	<0.001
Year = 2006	-0.214	0.039	<0.001
Year = 2007	-0.247	0.039	<0.001
Year = 2008	-0.014	0.037	0.707
Year = 2009	-0.059	0.037	0.111
Year = 2010	-0.049	0.037	0.188
Year = 2011	-0.023	0.037	0.540
Year = 2012	-0.455	0.041	<0.001
Degree of curve	0.039	0.001	<0.001
Shoulder width (ft)	-0.068	0.006	<0.001
Passing zone present	-0.112	0.026	<0.001
Centerline rumble strips	0.146	0.023	<0.001
Shoulder rumble strips	-0.027	0.036	0.453
Roadside hazard rating = 4 or 5	0.022	0.052	0.681
Roadside hazard rating = 6 or 7	0.096	0.055	0.083
Constant	-4.865	0.114	<0.001
Overdispersion parameter (α)	0.516	0.041	<0.001

Note: * represents pseudo r-square = 0.0699.

Table 90. Estimated SPFs for nighttime crashes.*

Variable	Coef.	S.E.	P-Value
Natural log of AADT	0.730	0.016	<0.001
Natural log of length	0.778	0.019	<0.001
Posted speed 50-55 mph	-0.035	0.026	0.182
Year = 2005	-0.864	0.059	<0.001
Year = 2006	-0.124	0.048	0.010
Year = 2007	-0.224	0.049	<0.001
Year = 2008	-0.033	0.047	0.490
Year = 2009	-0.161	0.049	0.001
Year = 2010	-0.090	0.048	0.060
Year = 2011	0.006	0.047	0.894
Year = 2012	-0.491	0.053	<0.001
Degree of curve	0.037	0.001	<0.001
Shoulder width (ft)	-0.062	0.007	<0.001
Passing zone present	-0.045	0.032	0.163
Centerline rumble strips	0.108	0.029	<0.001
Shoulder rumble strips	-0.058	0.046	0.204
Constant	-6.746	0.133	<0.001
Overdispersion parameter (α)	1.690	0.110	<0.001

Note: * represents pseudo r-square = 0.0747.

Table 91. Estimated SPFs for nighttime ROR crashes.*

Variable	Coef.	S.E.	P-Value
Natural log of AADT	0.621	0.017	<0.001
Natural log of length	0.784	0.021	<0.001
Posted speed 50-55 mph	-0.050	0.029	0.090
Year = 2005	-0.925	0.068	<0.001
Year = 2006	-0.095	0.054	0.077
Year = 2007	-0.224	0.055	<0.001
Year = 2008	-0.003	0.053	0.949
Year = 2009	-0.159	0.055	0.004
Year = 2010	-0.044	0.053	0.406
Year = 2011	-0.011	0.053	0.842
Year = 2012	-0.488	0.060	<0.001
Degree of curve	0.038	0.001	<0.001
Shoulder width (ft)	-0.068	0.008	<0.001
Passing zone present	-0.084	0.037	0.024
Centerline rumble strips	0.124	0.033	<0.001
Shoulder rumble strips	-0.086	0.053	0.104
Constant	-6.101	0.145	<0.001
Overdispersion parameter (α)	1.787	0.146	<0.001

Note: * represents pseudo r-square = 0.0634.

Table 92. Estimated SPFs for nighttime fatal and injury crashes.*

Variable	Coef.	S.E.	P-Value
Natural log of AADT	0.707	0.021	<0.001
Natural log of length	0.777	0.025	<0.001
Posted Speed 50-55 mph	-0.037	0.035	0.294
Year = 2005	-0.791	0.079	<0.001
Year = 2006	-0.038	0.064	0.548
Year = 2007	-0.269	0.067	<0.001
Year = 2008	-0.044	0.064	0.488
Year = 2009	-0.175	0.066	0.008
Year = 2010	-0.049	0.064	0.449
Year = 2011	-0.053	0.064	0.409
Year = 2012	-0.468	0.072	<0.001
Degree of Curve	0.036	0.002	<0.001
Shoulder width (ft)	-0.055	0.009	<0.001
Passing zone present	-0.023	0.044	0.601
Centerline rumble strips	0.144	0.039	<0.001
Shoulder rumble strips	-0.120	0.063	0.056
Constant	-7.249	0.179	<0.001
Overdispersion parameter (α)	1.948	0.212	<0.001

Note: * represents pseudo r-square = 0.0659.

The results of the estimates in each of the SPFs are consistent with engineering intuition. The estimates for both the natural logarithm of average annual daily traffic (AADT) and natural logarithm of curve length are consistent with values that have been found in previous research and that are included in the first edition of the American Association of State Highway and Transportation Officials' (AASHTO) Highway Safety Manual (HSM). (AASHTO 2010) Posted speed limits of 50-55 mph were found to be associated with lower crash frequencies, consistent with previous research (Wood and Porter 2013, Wood et al. 2015, Wood et al. 2016) and likely due to application of more forgiving geometric design criteria on high-speed roads relative to low-speed roads. The SPFs also indicate that, as the degree of curve increases (curve radius decreases), expected crash frequency is expected to increase. As shoulder width increases, the expected number of crashes decreases (consistent with the HSM). Horizontal curves with passing zones were found to be associated with fewer crashes, likely due to having longer available sight distances. The findings for passing zones are also consistent with the HSM. Centerline rumble strips were associated with an increase in expected crash frequency along horizontal curves, which is consistent with previous research. (Torbic et al. 2009) Shoulder rumble strips were associated with a decrease in expected crashes, which is also consistent with previous research. (Pitale et al. 2009) The effects of the year indicator variables accounted for differences in factors that change over time, such as differences in weather, crash reporting, and the driving population. The baseline year for all models was 2013.

The SPFs shown table 87, table 88, table 89, table 90, table 91, and table 92 were used in the empirical Bayes estimation procedure to develop CMFs. The values from the analysis (i.e., $\sum N_{Obs,after}$, $\sum N_{EB,after}$, $\sum VAR(N_{EB,after})$), as well as the estimated CMFs and 95-percent confidence intervals for each crash type and severity are provided in table 93 and table 94. As shown, the CMFs for total crashes and fatal plus injury crashes are statistically significant at the 95-percent confidence level while the other crash types and severities are not. However, all CMFs indicate a decrease in crashes due to the installation of horizontal curve warning pavement markings. The magnitudes of these reductions are 34.8 percent for total crashes, 30.7 percent for fatal and injury crashes, 23.1 percent for ROR crashes, 29.2 percent for nighttime crashes, 25.5 percent for nighttime ROR crashes, and 22.9 percent for nighttime fatal and injury crashes. As a result, horizontal curve pavement marking warnings appear to be a low-cost treatment that is effective in reducing crashes at horizontal curves.

Table 93. EB analysis results.

Crash Type	Total Crashes	Fatal + Injury Crashes	ROR Crashes
θ	0.652	0.693	0.769
$Stdev(\theta)$	0.115	0.144	0.140
θ 95% lower bound	0.426	0.411	0.495
θ 95% upper bound	0.877	0.975	1.044
$\sum N_{Obs,after}$	62	38	52
$\sum N_{EB,after}$	93.652	53.848	66.578
$\sum VAR(N_{EB,after})$	141.138	53.527	66.513

Table 94. EB analysis results cont.

Crash Type	Nighttime Crashes	Nighttime ROR Crashes	Nighttime Fatal + Injury Crashes
θ	0.708	0.745	0.771
$Stdev(\theta)$	0.157	0.176	0.211
θ 95% lower bound	0.400	0.399	0.358
θ 95% upper bound	1.015	1.090	1.185
$\sum N_{Obs,after}$	29	24	17
$\sum N_{EB,after}$	40.309	31.714	21.632
$\sum VAR(N_{EB,after})$	26.418	16.270	8.721

CONCLUSIONS

This chapter of the report assessed the efficacy of installing horizontal curve warning pavement markings in reducing crashes along horizontal curves. For the analysis, 263 treated curves from Pennsylvania on two-lane rural highways were used in an EB observational before-after study to develop CMFs. Nine years of data were used, covering the period from 2005 to 2013. The reference group used to develop SPFs included 21,902 curves on two-lane rural roads in Pennsylvania. CMFs were developed for total crashes, fatal and injury crashes, ROR crashes, nighttime crashes, nighttime ROR crashes, and nighttime fatal and injury crashes.

The results of the CMFs indicated that horizontal curve warning pavement markings are associated with reductions of 34.8 percent for total crashes, 30.7 percent for fatal and injury crashes, 23.1 percent for ROR crashes, 29.2 percent for nighttime crashes, 25.5 percent for nighttime ROR crashes, and 22.9 percent for nighttime fatal and injury crashes. The results were statistically significant at the 95-percent confidence level for the total crashes as well as for the fatal and injury crashes. Thus, the horizontal curve pavement warning markings are effective at reducing total crashes along horizontal curves and should be considered a low-cost safety improvement for horizontal curves on two-lane rural roadways. A limitation of the present study is that horizontal curve pavement markings will deteriorate over time. This analysis assumes that the pavement markings retain their visibility once applied at the treatment locations. Future research should consider how the pavement marking visibility changes over time when estimating the SPFs and CMFs in a before-after evaluation.

CHAPTER 8. CONCLUSIONS AND FUTURE RESEARCH NEEDS

Three evaluations were performed in the present study in order to supplement the existing published literature on roadway departure crashes along horizontal curves of two-lane rural highways. The findings from these evaluations are as follows:

1. The expected number of roadway departure crashes along a horizontal curve changes as a function of curve radius, radii of adjacent curves, and the upstream and downstream tangent lengths. Sharper upstream and downstream horizontal curves are associated with fewer expected roadway departure crashes based on the design consistency evaluations in the present study. Longer approach tangents are associated with a higher expected frequency of roadway departure crashes, while shorter upstream and downstream are associated with fewer expected roadway departure crashes.
2. The expected number of roadway departure crashes on horizontal curves increase as the friction demand increases. While the posted speed limit was used as a substitute for the design speed in the point-mass model for this evaluation, the findings suggest that, when the superelevation of a horizontal curve is fixed, higher operating speeds are associated with higher expected roadway departure crash frequencies.
3. Guiderail with delineators are expected to reduce total, fatal plus injury, run-off-road (ROR), and nighttime crashes along horizontal curves that are four degrees or sharper. On horizontal curves that are flatter than four degrees, guiderail with delineators are expected to reduce total, fatal plus injury, and nighttime crashes, but the present study did not find a benefit for ROR crashes on flatter curves with guiderail and delineators.
4. When applying guiderail only to horizontal curves on two-lane rural highways, the expected number of total and fatal plus injury crashes decreased in the present study on sharp curves, while the expected number of ROR and nighttime crashes increased on sharp curves. The presence of guiderail on flatter curves is expected to reduce total, fatal plus injury, and ROR crashes based on the results of the present study.
5. While an adequate sample of curves with delineators only could not be found from the roadway inventory database (RID), a crash modification factor (CMF) for delineators was derived based on the ratio of CMFs for guiderail with delineators (GD) and guiderail only (G only). On horizontal curves sharper than four degrees, delineators were found to reduce the expected number of fatal plus injury, ROR, and nighttime crashes. On flatter horizontal curves, however, delineators were associated with higher expected total, fatal plus injury, and ROR crashes.
6. Horizontal curve warning pavement markings are associated with fewer expected total, fatal plus injury, ROR, nighttime, nighttime ROR, and nighttime fatal plus injury crashes on two-lane rural highways.

There are several opportunities for future research in relation to roadway departure crashes along two-lane rural highways. These include, but are not limited to the following:

1. The relationship between superelevation deficiency and safety did not reveal a statistically significant finding in the present study. Future research should explore the effects of superelevation on roadway departure crashes.

2. This research did not include operating speeds or lane position data in the safety evaluations using the SHRP 2 RID data. Adding speed and lane position data to the design consistency-safety models estimated in the present study may further uncover relationships between the driver, roadway infrastructure, and safety.
3. The horizontal curves found in the RID data used in the present study did not identify an adequate sample of curves with only post-mounted delineators. Future research is recommended to identity curves that use only this treatment to offer guidance to motorists at night, and to perform a safety evaluation of this treatment.
4. Develop statistical models or crash frequency for pavement marking treatments that consider the visibility (i.e., retroflectivity) of the markings. The visibility of pavement marking countermeasures changes over time, and these trends should be integrated into future countermeasure evaluations.
5. The relationship between operating speed and crash frequency is not well-established. Efforts to better understand this may be uncovered in the Federal Highway Administration (FHWA) Rural Road Safety Initiative, but most transportation data analytics platforms (e.g., INRIX, iPEMS, etc.) collect data from probe vehicles, and may not reflect the entire traffic stream.
6. The relationship between safety and several geometric design features along horizontal curves are not yet well established. While there are CMFs in the first edition of the Highway Safety Manual (HSM) for lane width, shoulder width, horizontal curve radius, superelevation, vertical grade, driveway density, and roadside hazard rating, the interaction effects are not addressed through empirical research.
7. The in-service performance of roadside hardware is an emerging field, which offers opportunity to study the relationship between roadway departure events and longitudinal barrier systems.
8. While the FHWA has developed minimum retroreflectivity levels for signs and pavement markings based on driver visibility needs, the correlation between visibility levels and roadway departure crashes is not fully-developed.

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