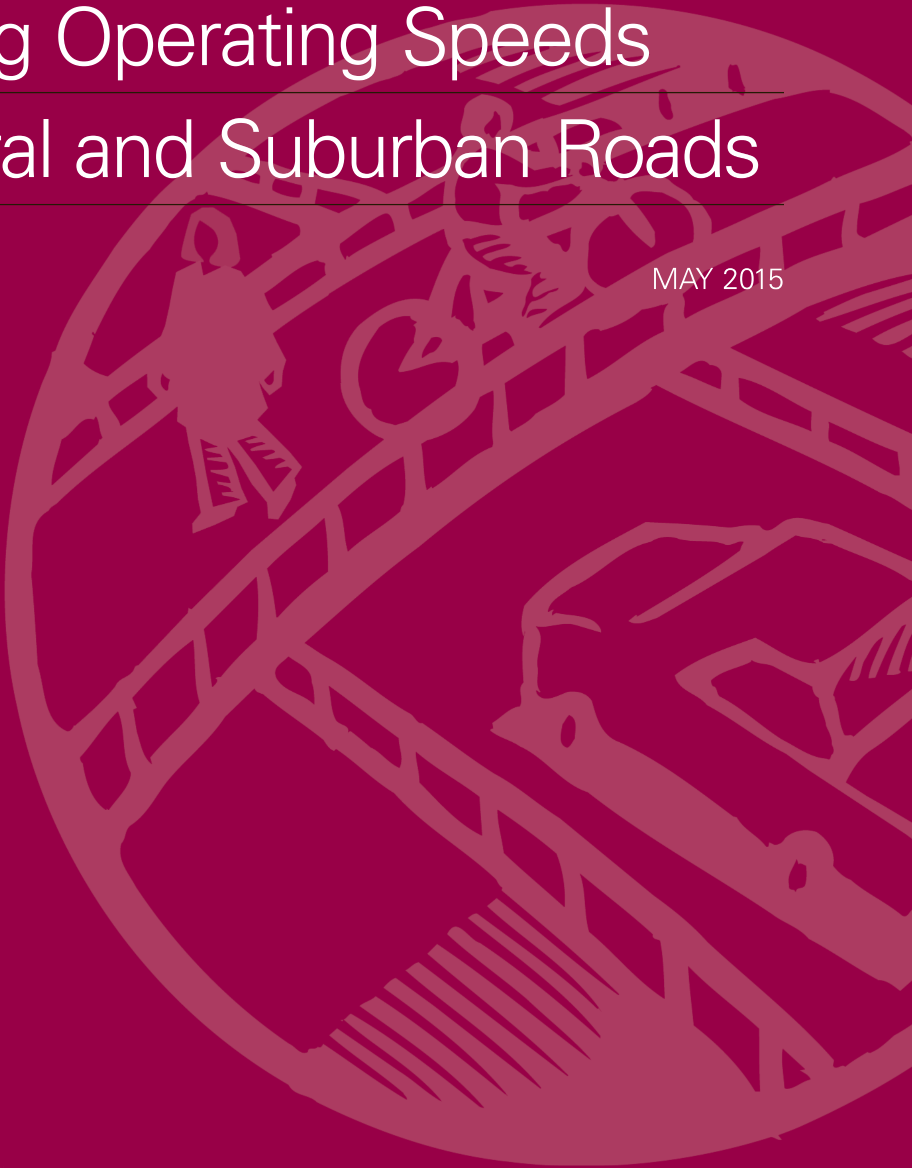
Factors Influencing Operating Speeds and Safety on Rural and Suburban Roads



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Turner-Fairbank Highway Research Center
6300 Georgetown Pike
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FOREWORD

The overall goal of the Federal Highway Administration's Speed Management Program is to improve the safety of the Nation's highways through the reduction of speeding and speed-related crashes. Drivers who exceed the speed limit or drive too fast for the conditions are involved in nearly one-third of all fatal crashes. Each year, more than 13,000 people are killed in speeding-related crashes. The majority of speeding-related crashes occur on roads that are not part of the interstate system. Local streets and collectors have the highest speeding-related fatality rate on a per vehicle miles driven basis. The challenge facing the safety professional is to design roadways so that drivers better understand the nature of the roadway and adjust their speed appropriately. Design guidance is needed so that roadways are designed and/or retrofitted to induce drivers to drive at more appropriate speeds.

This report documents the component factors affecting speed and safety on rural and suburban roadways that are not limited access. The report also describes the treatments that have the potential to reduce speed-related crashes.

Monique Evans
Director, Office of Safety
Research and Development

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Leverson Boodlal, P.E; Eric T. Donnell, Ph.D., P.E; Richard J. Porter, Ph.D., P.E; Dileep Garimella; Thanh Le; Kevin Croshaw; Scott Himes, Ph.D.; Philip Kulis; Jonathan Wood

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KLS Engineering, LLC
45155 Research Pl, Suite 200
Ashburn, VA 20147

Thomas D. Larson Pennsylvania Transportation Institute
The Pennsylvania State University
201 Transportation Research Bldg
University Park, PA 16802

University of Utah
201 Presidents Circle
Room 201
Salt Lake City, UT 84112

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16. Abstract
The objective of this project was to develop a technical report that describes treatments that result in driver self-selection of appropriate operational speeds on curve and tangent sections. The study was conducted in two phases**.** The first phase included a review of literature on design features and current practices associated with safer operating speeds and identification of treatments for field evaluations. The second phase involved evaluating treatments to determine their effectiveness in reducing speeds on two-lane horizontal curves in rural and suburban areas. High-friction surface treatment was evaluated at four treatment sites and three control sites in West Virginia. The speed and encroachment analyses found no consistent differences between the before and after time periods. The friction analysis, however, clearly demonstrated that the friction supply increased. Optical speed bars (OSB) were implemented and evaluated at seven sites in Massachusetts, four sites in Arizona, and eight sites in Alabama. Two different designs were tested as part of this research**,** and the results yielded inconsistent speed reductions at all the test sites. Based on the results, it can be concluded that the OSB designs used in this research were unsuccessful in reducing vehicle speeds. The safety effects of lane-width–shoulder-width combinations on rural two-lane, two-way road segments in Minnesota and Illinois were also estimated as part of this study. Parameters for lane width indicators showed that, with shoulder width ignored, the expected number of total (i.e., all types and severities) crashes increases as lane width decreases, but it is difficult to distinguish the performance of an 11-ft lane width from that of a 12-ft lane width. The main effect of shoulder width was a decrease in the expected number of crashes as shoulder width increased. In addition, the interaction of the lane width indicator and shoulder width showed that shoulder width has the greatest effect on safety when the lane width equals 10 ft. Shoulder width also has a greater effect on safety when the lane width is 11 ft than when the lane width is 12 ft.

17. Key Words
Horizontal curves, speeding related crashes, high friction surface treatment, optical speed bars, lane-width–shoulder-width combinations

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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

AASHTO	American Association of State Highway and Transportation Officials
ALDOT	Alabama Department of Transportation
ANOVA	Analysis of Variance
CMF	Crash Modification Factors
CT	Circular Texture
DF	Dynamic Friction
FHWA	Federal Highway Administration
GIS	Geographic Information System
HCM	Highway Capacity Manual
HDM	Highway Data Management
HFST	High Friction Surface Treatment
HSIS	Highway Safety Information System
HSM	Highway Safety Manual
IFI	International Friction Index
KML	Keyhole Markup Language
MCPW	Mohave County Public Works
MM	Mile Marker
MPD	Mean Profile Depth
MPH	Miles per Hour
MUTCD	Manual on Uniform Traffic Control Devices
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NJDOT	New Jersey Department of Transportation
OSB	Optical Speed Bars
P.E.	Professional Engineer
PC	Point of Curvature
PDO	Property Damage Only
PennDOT	Pennsylvania Department of Transportation
PI	Point of Intersection
PSL	Posted Speed Limit
PT	Point of Tangent
PTOE	Professional Traffic Operations Engineer
RSA	Road Safety Audit
SD	Standard Deviation
SDF	Severity Distribution Function
SN	Skid Number
SRPEDD	Southeast Regional Planning and Economic Development District
TRB	Transportation Research Board
TWLTL	Two-Way Left-Turn Lane
VBA	Visual Basic for Applications
WVDOT	West Virginia Department of Transportation

CHAPTER 1. INTRODUCTION

United States traffic safety statistics show that fatalities resulting from motor vehicle crashes decreased from 2005 through 2011, but increased slightly in 2012. Despite these encouraging trends, the National Highway Traffic Safety Administration's (NHTSA) Fatality Analysis Reporting System reported approximately 5.6 million traffic crashes in 2012, including 33,561 fatalities and more than 2.3 million injuries.(1) Rural fatalities accounted for 54 percent (18,170) of all traffic fatalities in 2012, although 19 percent of the U.S. population lived in rural areas. Further, in 2012, the fatality rate per 100 million vehicle mi traveled was 2.4 times higher in rural areas than in urban areas. This disparity between rural and urban road fatal crash rates has not changed appreciably in more than a decade.(2)

NHTSA defines speeding-related crashes as "the driver behavior of exceeding the posted speed limit (PSL) or driving too fast for conditions," which has been cited as a contributing factor in approximately 31 percent of traffic fatalities in the United States.(3) This same research shows that a significant percentage of crashes occurring at night and on rural roads are speeding related. Speeding-related factors were also prevalent among single-vehicle run-off-road crashes and crashes on horizontal curves. It should be noted that speeding-related crash statistics that are compiled and reported annually by NHTSA rely on databases consisting of electronically coded police accident reports. Therefore, an element of subjectivity (i.e., the reporting officer's expert opinion of whether speed was a contributory cause of a crash) is present. The extent of reporting bias is unknown.

The relationship between operating speeds and safety is not entirely clear. There is no consensus regarding the association between speed and crash frequency; however, there is agreement that crash outcomes will be more severe as speed increases. Therefore, it is important to consider how roadway and roadside features influence driver speed choice. Roadway and roadside features provide cues to drivers that influence their speed selections. Supplemental measures, such as traffic control devices and speed management strategies, are sometimes used to influence driver speed selections to be more consistent with target operating speeds. Designing a roadway to influence drivers to travel at a particular "target speed," while discouraging them from traveling at an excessive or inappropriate speed, may help to prevent speeding-related crashes or reduce their severity when they do occur. In such cases, free-flow operating speeds near design speeds and posted speeds are likely correlated with community goals and expectations. The Federal Highway Administration's (FHWA) *Speed Concepts: Informational Guide* reviewed a number of speed management practices and technologies and describes speed-based technical processes that often lead to operating speeds higher than design speeds and PSLs.(4) The FHWA suite of speed management resources also includes *Engineering Countermeasures for Reducing Speeds: A Desktop Reference of Potential Effectiveness* and *Speed Management: A Manual for Local Rural Road Owners*.(5,6)

This report builds on these previous efforts by further exploring technical speed-related design processes and their speed outcomes as well as highway and traffic engineering features that influence operating speeds and safety on rural and suburban roads.

SCOPE OF RESEARCH AND REPORT ORGANIZATION

The objective of this project was to develop a technical report that describes features and treatments that influence operating speeds on curve and tangent sections of rural and suburban roads. The research team gathered information for the report from a review and critical synthesis of published literature as well as new research conducted as part of this project.

The remainder of this report is organized into the following chapters:

- Chapter 2 provides background and the results of a synthesis of literature related to the characteristics of rural and suburban roads, speed-related outcomes of current design and operational practices, and traffic engineering treatments, as well as design and retrofit decisions and practices that influence operating speeds.

- Chapter 3 describes the process of defining the scope of the research conducted as part of this project.

- Chapter 4 describes an operational evaluation of the high-friction surface treatments (HFST) applied to horizontal curves on rural two-lane roads in West Virginia.

- Chapter 5 documents a speed evaluation of optical speed bar (OSB) treatments applied to rural and suburban road segments in Arizona, Alabama, and Massachusetts.

- Chapter 6 presents an observational, cross-sectional study on the safety effects of lane width and shoulder width combinations on rural two-lane, two-way road segments.

- Chapter 7 summarizes the entire research effort, provides general conclusions, and outlines general considerations for future studies.

CHAPTER 2. BACKGROUND AND LITERATURE REVIEW

This chapter consists of three main sections. The first section briefly describes characteristics of rural and suburban roads. The second section describes the speed-related outcomes of current design and operational practices—specifically, the discussion explains why intermediate and lower speed roads sometimes operate in a state of design, operating, and posted speed discord, as defined by the FHWA *Speed Concepts Guide*.[4] The third section identifies traffic engineering treatments, as well as design and retrofit decisions and practices, that influence operating speeds.

DEFINING RURAL AND SUBURBAN ROADWAYS

Roadway design criteria differ by functional classification, which includes a definition of area type—urbanized, small urban area, or rural.[7] U.S. Bureau of Census designates the urbanized areas as having populations of 50,000 or more. Small urban areas have populations of 5,000 or more and are not within urbanized areas.[7] State transportation department functional classification maps provide the boundaries that define the extent of both urbanized and small urban areas. Rural areas are all remaining areas in a State outside the urbanized and small urban area boundaries. Road designers consider whether an existing or planned road is urban or rural, with urban including both urbanized and small urban areas. State transportation department functional classification maps vary in terminology use but appear consistent with intent. For example, the Utah Department of Transportation delineates urban and small urban areas.[8] Washington State Department of Transportation shows city limits and urban boundaries.[9] Virginia Department of Transportation maps include boundaries for urbanized areas and small urban clusters.[10] New Jersey Department of Transportation (NJDOT) shows only one type of urban boundary.[11] Although different terms are used, the boundaries seem to be consistently based on the FHWA and U.S. Census Bureau definitions. This is not surprising because State functional classification requirements are tied to the FHWA Federal-aid Program.[7]

The preceding discussion demonstrates that by using State transportation department functional classification maps, any existing or planned road can be designated as urban or rural. Suburban road designations are not as definite. The general definition of suburban is "the residential area on the outskirts of a city or large town," but land uses can be more than just residential.[12] The U.S. Department of Justice defines "suburban" as a census block group no more than 30 mi from an urban area with a density of at least 500 people per mi^2 but fewer than 2,000 people per mi^2.[13] Ban and Ahlquist defined four geographic areas of cities—urban zone, urban cluster, suburban, and exurban.[14] They defined suburban areas as those areas located between urban clusters and exurban areas, but with blurred boundaries. They define these areas as follows:

- **Urban cluster**: census block groups of at least 2,500 people but less than 50,000 people.
- **Suburban**: non-central county classified as metropolitan.
- **Exurban**: metropolitan counties outside the ring of suburban counties.

The U.S. Department of Agriculture describes a rural area as countryside or settlements of fewer than 2,500 people, implying that the areas between urban boundaries and rural areas with populations greater than 2,500 may be considered suburban.[15]

Some published documents have tried to summarize common physical characteristics of suburban areas. The *FHWA Urban Boundary and Federal Functional Classification Handbook* provides general arterial spacing observations by area type: ½ to 1 mi in urban areas, 1 to 2 mi in suburban areas, and 2 to 3 mi in low-density areas.[16] The FHWA publication *Access Management in the Vicinity of Intersections* describes suburban areas as consisting of large-scale residential, commercial, industrial, or retail developments typically separated by larger distances than in urban areas.[17] The following characteristics describe suburban areas:

- Medium-to-long block lengths, 400 ft to 1 mi.
- Signalized intersections on arterials and major collectors.
- Speeds from 35 to 55 miles per hour (mph).
- 30,000 to 50,000 vehicles per day on mainline roadways.
- 5,000 to 15,000 vehicles per day on side streets and nonresidential driveways.
- Moderate-to-large setback of structures.
- Non-traversable medians or continuous two-way left-turn lanes (TWLTL).
- Left and right turn lanes.
- Six or fewer traffic signals per mi.

The FHWA *University Course on Bicycle and Pedestrian Transportation* offers the idea that three types of land use imply suburban character:[18]

1. Individual tract subdivisions, planned as units, with a sense of order derived from in-road systems. The units consist mostly of single-family, residential homes. Some warehousing, shopping, and medical developments may also fall into this category.

2. A linear arterial that is the main source of a community's image. No organized land-use planning occurred, and the arterial serves a mix of long trips and local business transactions.

3. Bypassed land that was skipped during the initial development of the area for cheaper land and was filled in later with a mix of land uses.

The Texas Department of Transportation's *Roadway Design Manual* defines a suburban roadway as a high-speed roadway that serves as a transition between urban streets and high-speed, rural highways.[19] It is typically 1 to 3 mi long, has light-to-moderate driveway densities (10 to 30 driveways per mi), and has both rural and urban characteristics. The specific mixed characteristics identified were high-speed operations while using curb and gutter for drainage.

The Pennsylvania Department of Transportation's (PennDOT) and NJDOT's *Smart Transportation Guidebook* defines the following seven "context areas" from least to most developed:[20]

1. Rural.
2. Suburban neighborhood.
3. Suburban corridor.
4. Suburban center.
5. Town/village neighborhood.
6. Town center.
7. Urban core.

The PennDOT and NJDOT guidebook includes a set of quantifiable characteristics for each of the seven context areas and a recommendation to base identified land use on this information. Table 1 summarizes the quantifiable characteristics.

Table 1. Quantifiable characteristics of land use contexts (PennDOT and NJDOT, 2008).[(20)]

Characteristic	Rural	Suburban Neighborhood	Suburban Corridor	Suburban Center	Town/Village Neighborhood	Town Center	Urban Core
Density Units (DU)[1]	1 DU/ 20 ac	1–8 DU/ac	2–30 DU/ac	3–20 DU/ac	4–30 DU/ac	8–50 DU/ac	16–75 DU/ac
Building Coverage	N/A	< 20 percent	20–35%	35–45%	35–50%	50–70%	70–100%
Lot Size/Area	20 ac	5,000–80,000 ft^2	20,000–200,000 ft^2	25,000–100,000 ft^2	2,000–12,000 ft^2	2,000–20,000 ft^2	25,000–100,000 ft^2
Lot Frontage[2]	N/A	50–200 ft	100–500 ft	100–300 ft	18–50 ft	25–200 ft	100–300 ft
Block Dimensions	N/A	400 ft wide by varies	200 ft wide by varies	300 ft wide by varies	200 by 400 ft	200 by 400 ft	200–400 ft
Maximum Height	1–3 stories	1.5–3 stories	1 story retail; 3–5 story office	2–5 stories	2–5 stories	1–3 stories	3–60 stories
Minimum/ Maximum Setback	Varies	20–80 ft	20–80 ft	20–80 ft	10–20 ft	0–20 ft	0–20 ft

[1]The guidebook does not define a density unit and may instead refer to a dwelling unit; dwelling units per acre are used in the guidebook to define high-, medium-, and low-density areas.
[2]The distance measured between points where side property lines meet road right-of-way lines.
N/A = Not Applicable

Land use contexts should be broadly defined for road segments greater than 600 ft in length because of practical limitations on the frequency of changing the roadway typical section over a short stretch of road.[(20)]

The *Smart Transportation Guidebook* includes a "matrix of design values" with design criteria as rows and land use contexts as columns for five different roadway types: 1) regional arterial, 2) community arterial, 3) community collector, 4) neighborhood collector, and 5) local road.[(20)] This roadway typology is different than the existing functional classification system outlined by FHWA and was proposed to capture the actual role of the roadway in the surrounding community.[(7)] Access, mobility, and speed are considered on the road segment of interest as opposed to using only one functional classification for an entire highway.

The literature review on area type definitions demonstrated the lack of an objective definition for "suburban." Suburban areas may be inside or outside of urban boundaries. Physical characteristics vary. Because the scope of this research project is rural and suburban roads, the research team relied on State-specific definitions of these area types when gathering information about rural and suburban roads in that State.

SPEED CONCEPTS

This section describes the relationships among design, operating, and posted speed limits during the project development process (pre- to post-construction). In addition, it also discusses the known relationship between speed and safety, including both crash frequency and severity.

Relationship Among Design, Running, Operating, and Posted Speeds

This section discusses road geometric design practices and speed-related outcomes, with particular focus on the concepts of design speed, operating speed, and posted speed. Several sections draw significantly on the FHWA's *Speed Concepts: Informational Guide* and *Understanding Speed Concepts*, as well as on follow-up work to the informational guide published by Porter et al.(4,21,22) The following sections use both operating speed and running speed to describe relationships between design speed and observed speeds. Operating speed is "the speed at which drivers are observed operating their vehicles during free-flow conditions." Running speed is "the speed at which an individual vehicle travels over a highway section." Operating speeds and running speeds during off-peak, low-volume conditions provide similar insights on how drivers select speeds based on the road geometrics, particularly on uninterrupted flow facilities.

U.S. transportation agencies develop designs and prepare plans for road construction. Designers rely on a set of adopted standards and policies that include design criteria. The most commonly adopted policy is *A Policy on Geometric Design of Highways and Streets* (*Green Book*), published by the American Association of State Highway and Transportation Officials (AASHTO).(23) Design criteria in the *Green Book* are based on research and practice and are generally expressed as minimum, maximum, or ranges of values for design elements (e.g., minimum horizontal curve radius, maximum grade). When adopting or recommending design criteria, individual State transportation departments and other transportation agencies, as well as AASHTO, consider factors such as safety, efficiency, driver comfort, aesthetics, construction cost, and future maintenance activities. AASHTO updates design criteria in the *Green Book* as meaningful research results become available. The process a designer follows to establish design criteria for any given project remains essentially unchanged since the earliest design policies.(24,25)

Design Speed

U.S. road geometric design practice is based on selecting and applying a design speed. The design speed is usually selected during preliminary design activities and influences subsequent design decisions. Fitzpatrick et al. provided a synthesis of design speed selection practices.(26) Donnell et al. provided the inputs to and outcomes of design speed selection.(4)

A review of AASHTO design policies revealed three different design speed definitions:

- **Pre-1954**: The maximum appropriately uniform speed that probably will be adopted by the faster group of drivers but not, necessarily, by the small percentage of reckless ones.[27]

- **1954−2001**: The maximum safe speed maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern.[23]

- **2001–present**: A selected speed used to determine the various geometric design features of the roadway.[28, 29]

Although the wording has changed, the basic application and interpretation has not. Definitions and supplemental guidance on design speed selection imply the design speed is conceptually consistent with operating speeds at the higher end of the speed distribution observed on a road segment. In other words, the majority of drivers will travel at or below the design speed. The second implication is that drivers traveling at or below the design speed are traveling at safe speeds. Drivers traveling above the design speed also travel above the safe speed. Subsequent research and analysis have shown that traveling above the design speed is not necessarily less safe than traveling below the design speed. There are documented cases where the PSL is higher than the design speed but less than inferred design speed, the "maximum speed for which all critical design-speed-related criteria are met at a particular location."[21] There has been no thorough research regarding safety performance on road segments with different design speeds, inferred design speed, operating speed, and posted speed relationships.

As early as 1954 and 1957, AASHTO design policies described the expected relationships between the design speed and average running speed—and current policy reflects these. The policies suggested that running speeds would be close to design speeds when design speeds were low. It was also recognized that "some sections of low design speed highways are frequently overdriven, with an appreciable number of drivers exceeding the design speed." [24] Design policy noted that the speed selected by most drivers would increase as design speed increased, but at a lower rate. AASHTO provides the following example[22]:

> Comparing the observed average speeds with calculated design speeds, it is found that on sections of highway having a 30-mph design speed the average running speed is approximately 90 percent of the design speed. The ratio gradually decreases to about 70 percent for highway sections with a design speed of 70 mph. (p. 40)

AASHTO expanded the design speed and running speed relationships to include numbers for both low volume and peak volumes.[24] Figure 1 illustrates these relationships.[22] Table 2 provides current design speed–running speed relationships.

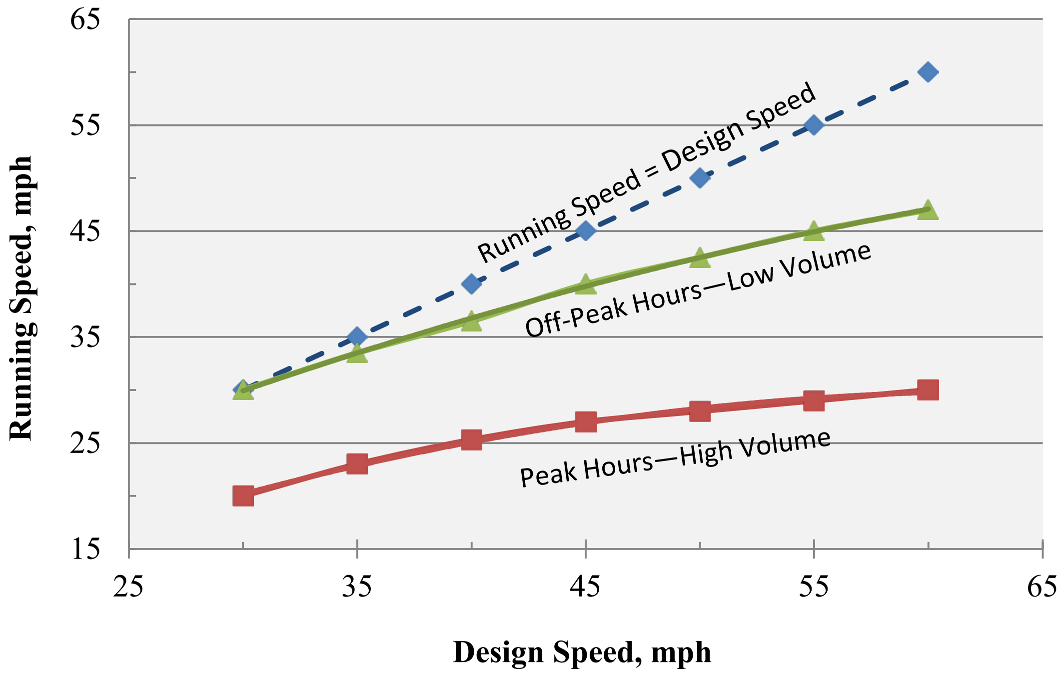

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Figure 1. Graph. Approximate relationships between design and running speeds for urban conditions (figure 1 from Porter et al., 2012).[(22)]

Table 2. Relationship between design speed and average running speed (table 3-6 in AASHTO 2011).[(28)]

Design Speed (mph)	Average Running Speed (mph)	Design Speed (mph)	Average Running Speed (mph)
15	15	50	44
20	20	55	48
25	24	60	52
30	28	65	55
35	32	70	58
40	36	75	61
45	40	80	64

Figure 1 and table 2 illustrate that current practice related to selecting design speed continues to be influenced by early design speed definitions and the ideal design speed and running speed relationships. The early definitions suggest selecting a high design speed if a majority of the drivers will select speeds below the design speed and the design speed reflects a maximum safe speed. Operating speeds will likely be close to their targeted range because they are not expected to increase at a rate directly proportional to design speed. There will also be a larger buffer between operating speeds and design speeds at higher design speed values, which is desirable because (at the time) design speed represented the maximum safe speed. However, the design speed definition has changed. The current definition removes direct references to safety but reflects the same basic philosophy in supplemental guidance related to design speed selection until recently[(29)]:

> Except for local streets...every effort should be made to use as high a design speed as practical to attain a desired degree of safety….(p. 67)

AASHTO states that "every effort should be made to attain a desired combination of safety, mobility, and efficiency within the constraints of environmental quality, economics, aesthetics, and social and political impacts."[28]

Once planners select a design speed, they then determine minimum (or maximum) design values for a number of elements. The next section discusses these other design criteria.

Other Design Criteria

Design criteria most directly related to design speed are for horizontal curvature (superelevation, side friction, and curve radius relationships) and required stopping sight distance. The maximum side friction factor and maximum rate of superelevation parameters are the parameters used to establish the minimum radius of curvature. The values for maximum side friction factor are based on driver comfort, not on physical side friction supply and demand relationships. The result is a significant "margin of safety" between friction values used for design and friction supply at the road surface–tire interface at the point of impending skid. Figure 2 provides an example.[22] The numbers for side friction supply were based on a recent re-analysis of findings by Harwood et al.[30] They are applicable for roadways on level or near-level grades; National Cooperative Highway Research Program (NCHRP) Project 15-39 is currently investigating aspects of friction supply within horizontal curves on steep grades.[31] Figure 3 illustrates the effect of the margin of safety on determining the minimum radius of curve.[22] An additional relationship provided shows the minimum radius of curve based on truck rollover thresholds for trucks traveling at the design speed. Harwood et al. also provides the basis for the rollover threshold numbers.[30] Side friction demand is one factor used to determine horizontal curve signing needs. Bonneson et al. provided a detailed review of related published literature and curve signing guidelines.[32]

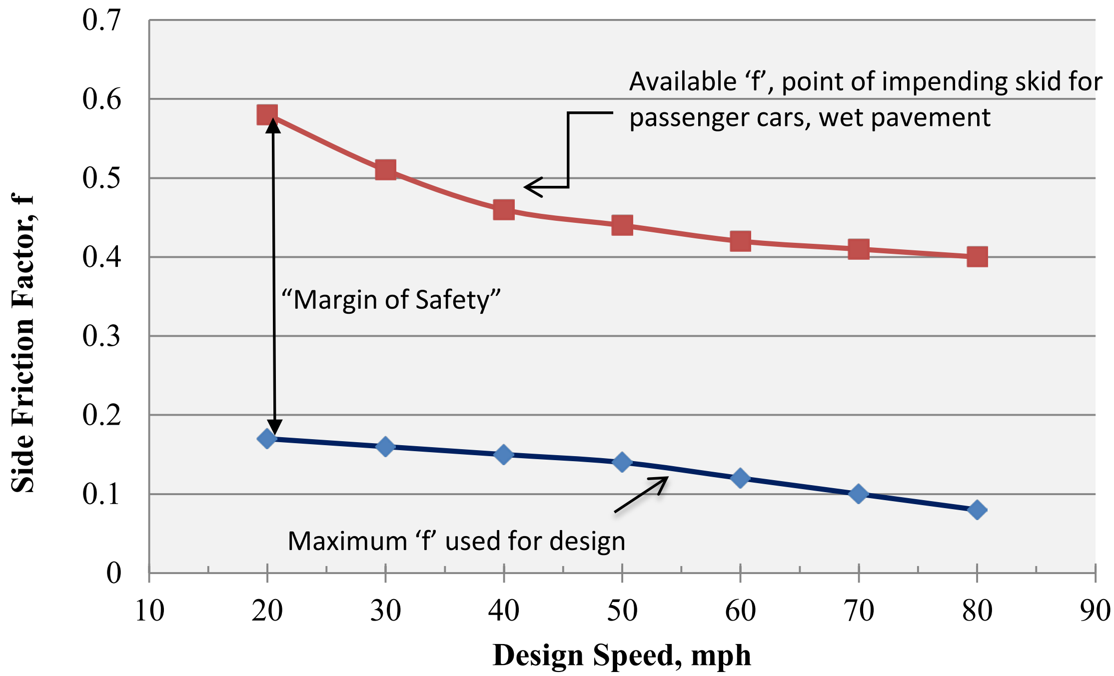

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Figure 2. Graph. Comparison of maximum side friction factor used for design to available side friction (figure 2 from Porter et al., 2012).[(22)]

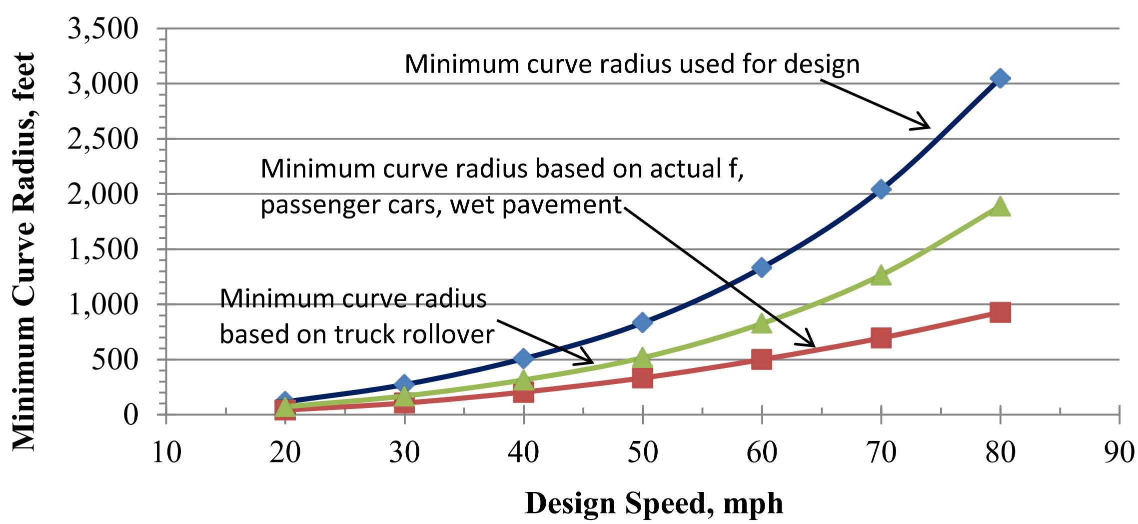

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Figure 3. Graph. Comparison of minimum curve radius used for design to minimum curve radii (figure 3 from Porter et al., 2012).[(22)]

The perception-reaction time and the deceleration rate supply the parameters used to establish the minimum required stopping sight distance. The currently recommended design is a perception-reaction time of 2.5 s.[28] The 2.5-s perception-reaction time is based on a synthesis of four studies and is believed to encompass the capabilities of most drivers, including older drivers, and to exceed the 90th percentile of reaction time for all drivers. (See reference 28 and 33 through 36.) A deceleration rate of 11.2 ft/s is used for design.[37] This value is based on research by Fambro et al.[36] Most drivers decelerate at rates greater than 14.8 ft/s and approximately 90 percent decelerate at rates greater than 11.2 ft/s. These rates are well within those required for drivers to maintain steering control during braking on wet surfaces.[28]

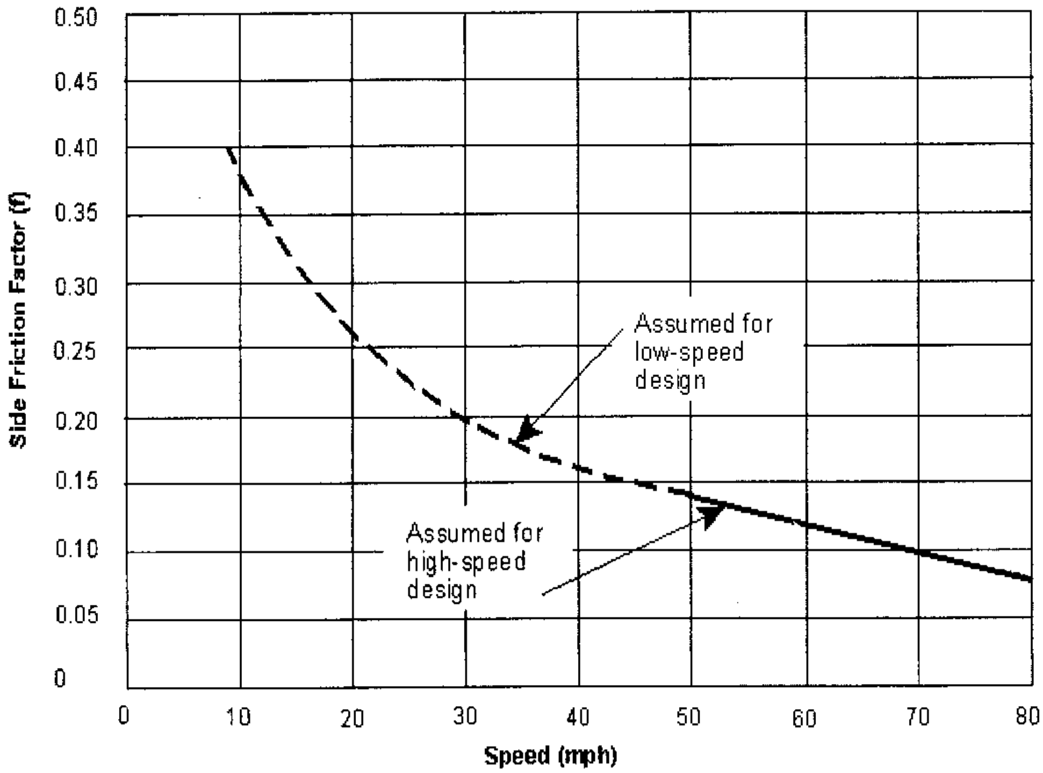
A conservative design approach becomes apparent when comparing the limiting parameter values used for the design of horizontal curves and calculating required stopping sight distance to the observed parameter values found in published research results. Friction supply is significantly greater than maximum friction used for design. The perception-reaction time used for design represents a 90th percentile value (i.e., 90 percent of drivers have faster perception-reaction times). Deceleration rates assumed for design represent a 10th percentile value (i.e., 90 percent of drivers decelerate at higher rates). The conservative approach is consistent with other engineering disciplines when there is variability in design parameters. Values suggest that a majority of drivers can traverse a horizontal curve or stop prior to hitting an object in the roadway if they are traveling at a rate higher than the design speed—even if the minimum design values have been used.

Actual design values are determined once minimum values are established. The *Green Book* recommends that “Above-minimum design criteria for specific design elements should be used, where practical…” and “Although the selected design speed establishes the limiting values of curve radius and minimum sight distance that should be used in design, there should be no restriction on the use of flatter horizontal curves or greater sight distances where such improvements can be provided as part of an economical design.”[28] The process results in the inferred design speed, defined above as meeting the maximum speed for which all critical design speed-related criteria at a particular location are greater than the design speed. Speed-related road cues perceived by the driver (e.g., available sight distance, degree of curvature) are more associated with inferred design speed than with design speed. Operating speeds have been shown to increase as inferred design speed increases.[21,38,39] The following example illustrates some of the key concepts discussed here.

Example

A suburban road is classified as an urban collector. The design speed is 35 mph and the maximum rate of superelevation is 6 percent. The road is undivided with a normal crown of 2 percent.

The maximum side friction factor for a 35 mph design speed is 0.18 (see figure 4).



Source: AASHTO

Figure 4. Graph. Maximum side friction factor assumed for design (figure 3-6 from AASHTO 2011).(28)

The equation in figure 5 then computes the minimum radius for horizontal curves on this road:

$$R_{\min} = \frac{V^2}{15(0.01e_{\max} + f_{\max})}$$

Figure 5. Equation. Minimum radius of horizontal curvature.

Where:

$R_{\min}$ = minimum radius of horizontal curvature (ft).
V = design speed (mph).
$e_{\max}$ = maximum rate of superelevation (percent).
$f_{\max}$ = maximum side friction factor.

The calculation to determine minimum radius of horizontal curvature for the suburban road in this example, with a 35-mph design speed, 6-percent maximum rate of superelevation, and 0.18 maximum side friction factor, is 340 ft.

Assume a radius of 1,000 ft is selected for a curve on this suburban road based on the *Green Book* guidance that “there should be no restriction on the use of flatter horizontal curves or greater sight distances where such improvements can be provided as part of an economical design." [28]

The cross section would remain at normal crown for this selected radius if superelevation and side friction are distributed according to Method 2 (see table 3).

Table 3. Minimum radii and superelevation for low-speed urban streets (part of figure 3-13b from AASHTO 2011).[28]

e (%)	V_d = 15 mph R (ft)	V_d = 20 mph R (ft)	V_d = 25 mph R (ft)	V_d = 30 mph R (ft)	V_d = 35 mph R (ft)	V_d = 40 mph R (ft)	V_d = 45 mph R (ft)
-6.0	58	127	245	429	681	1,067	1,500
-5.0	56	121	231	400	628	970	1,350
-4.0	54	116	219	375	583	889	1227
-3.0	52	111	208	353	544	821	1125
-2.8	51	110	206	349	537	808	1107
-2.6	51	109	204	345	530	796	1089
-2.4	51	108	202	341	524	784	1071
-2.2	50	108	200	337	517	773	1055
-2.0	50	107	198	333	510	762	1039
-1.5	49	105	194	324	495	736	1000
0	47	99	181	300	454	667	900
1.5	45	94	170	279	419	610	818
2.0	44	92	167	273	408	593	794
2.2	44	91	165	270	404	586	785
2.4	44	91	164	268	400	580	776
2.6	43	90	183	265	396	573	767

Source: AASHTO

The inferred design speed in this case is the maximum speed that meets the side friction factor criterion for the horizontal curve. A superelevation value of -2 percent is used for the inferred design speed calculations because the adverse cross slope in one travel direction will result in a component of the vehicle's weight pointed to the outside of the curve. (The opposite travel direction would have a superelevation of +2 percent and, as a result, a higher inferred design speed.) The equation in figure 6 computes the required side friction factor for a vehicle traversing a horizontal curve:

$$f = \frac{V^2}{15R} - 0.01e$$

Figure 6. Equation. Required side friction factor.

Where:

f = required side friction factor.
V = speed of vehicle traversing the horizontal curve (mph).
R = radius of horizontal curvature (ft).
e = superelevation (percent).

The inferred design speed estimate for the horizontal curve uses an iterative process. Table 4 summarizes the results. Donnell et al. provides additional detail on how to perform inferred design speed calculations.(21)

Table 4. Results of iterative inferred design speed calculations.

Iteration	*V*	*f*	Comment
1	35	0.102	The required side friction, 0.102, is less than the maximum for 35 mph, 0.18. The inferred design speed is higher than 35 mph.
2	45	0.155	The required side friction, 0.155, is greater than the maximum for 45 mph, 0.15. The inferred design speed is lower than 45 mph.
3	40*	0.127	The required side friction, 0.127, is less than the maximum for 40 mph, 0.16. The inferred design speed is 40 mph.

*The inferred design speed in this example is calculated to the nearest 5-mph increment for which the design side friction is satisfied. If one were to assume a linear change in friction with each unit increase in the speed, the inferred design speed could be calculated to the nearest 1 mph, using the process described in this example.

The calculations used to populate table 4 were based on the maximum side friction factors used for design. As previously noted, the design values are significantly lower than limiting values of side friction (see figure 4). The curve in this example would have an inferred design speed of approximately 80 mph using the calculations based on side friction supply for passenger cars on wet pavement at the point of impending skid.

Speeding-Related Outcomes of U.S. Design Practice

Donnell et al. describes and illustrates the speed-related outcomes of U.S. road design practice. Figure 7 illustrates a typical outcome on low- to intermediate-speed roads.(21) While design speed is determined during the design process, inferred design speeds are determined implicitly (but typically not considered or calculated) as a result of geometric design decisions. The inferred design speeds are often higher than the design speed because designers are encouraged to exceed minimum values determined for geometric design features that are determined based on the design speed. The result is that many design features meet criteria for design speeds far greater than the design speed (shown by the inferred design speed line above the designated design speed line in figure 7). Initial posted speeds are generally equal to or less than the design speed.(26) As figure 7 shows, actual operating speeds may be higher than both the speed limit and the design speed after a road is open to traffic.

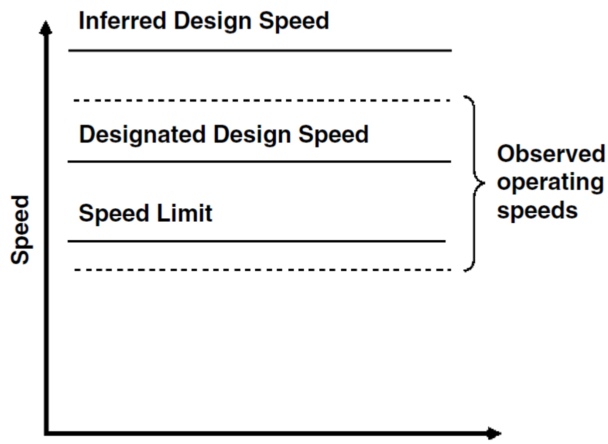


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Figure 7. Chart. Observed speed-related outcomes of the typical U.S. design practice (figure 2a from Donnell et al., 2009).[21]

Observations show that mean operating speeds tend to be higher than design speeds of approximately 45 to 50 mph or less (the exact crossing point appears to depend on facility type). Observed 85th percentile speeds tend to be higher than design speeds of approximately 55 mph or less. Figure 8 illustrates these findings.[22]

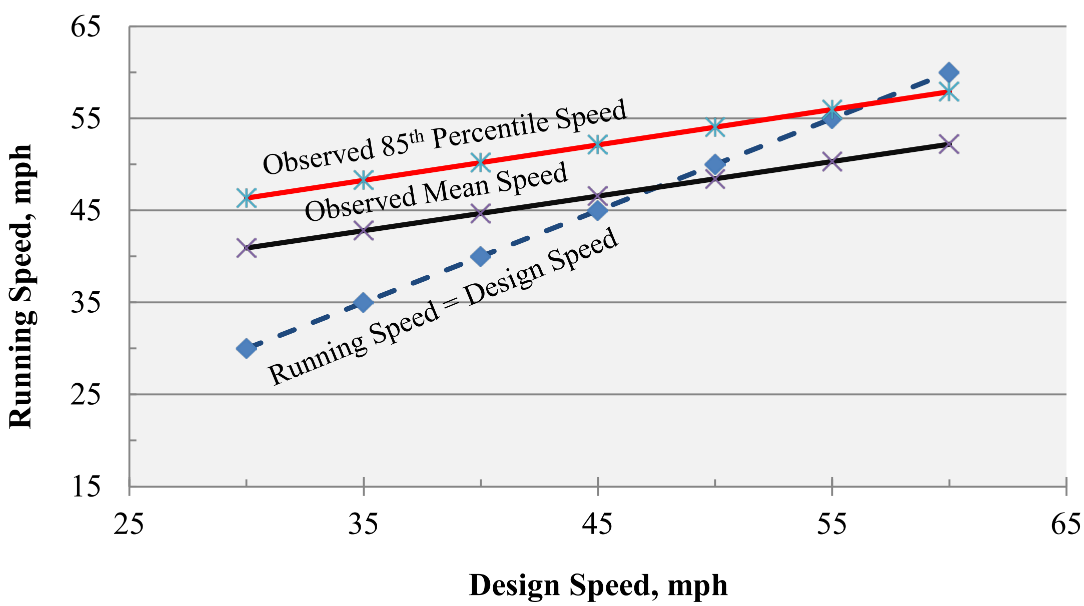

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Figure 8. Graph. Observed relationships between design speed and operating speeds (figure 5 from Porter et al., 2012).[22]

Tarris et al. first illustrated conceptually possible speed-related outcomes of U.S. road design practice on road segments with a series of tangents and curves (see figure 9).[40] The figure compiles many of the ideas of the preceding discussion. Donnell et al. later used a series of case studies with field data that invalidated the concepts portrayed in this figure.[4]

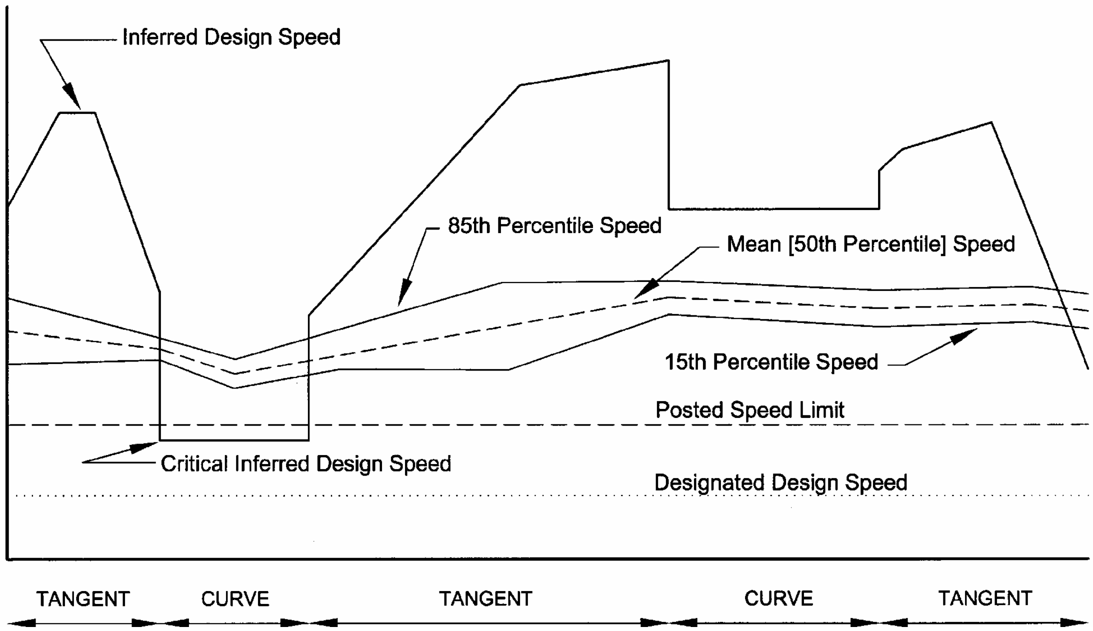

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Figure 9. Diagram. Conceptual speed-related outcomes of U.S. road design practice (adapted from figure 1 in Tarris et al., 2000).[40]

Donnell et al. defined the terms "speed harmony" and "speed discord."[4,21]

- **Speed harmony**: Condition resulting when the designated design speed is within a specified range (i.e., ± 5 mph) of the observed 85th percentile operating speed; the 85th percentile operating speed is within a specified range (i.e., ± 5 mph) of the PSL. The inferred design speed is equal to or greater than the designated design speed; the posted speed is less than or equal to the designated design speed.

- **Speed discord:** Roadway design that produces a condition in which the design speed is lower than the PSL, various operating speed measures, or both.

Design that results in speed harmony is generally desired.

Relationship Between Speed and Safety

Speeding and relationships between operating speed and safety are poorly understood, likely a result of confusing terminologies, inconsistent crash coding, and conflicting research results. Consider the following:

- Discussions that relate speed to safety sometimes reference studies that show a "u-shaped curve"—while faster vehicles have an increased crash risk, so do slower moving vehicles.[41,42]

- Annual compiled and reported speeding-related crash statistics rely on databases consisting of electronically coded police crash reports (e.g., Fatality Analysis Reporting System, General Estimates System, and other State crash databases). These data reflect an element of subjectivity (i.e., the reporting officer's expert opinion of whether speed was a contributory cause of a crash); however, the extent of this reporting bias is unknown.

- Speed is central to decisions made throughout the lifecycle of highways and streets. Agencies and units within agencies (e.g., design, traffic control, operations, enforcement) have different responsibilities and use unique terminologies to define speed (e.g., safe speed, design speed, inferred design speed, operating speed, regulatory speed, posted speed, *N*th percentile speed). Conclusions about whether traffic in general or a particular driver was "traveling too fast for conditions" would also likely vary between groups.

Two separate literature reviews on speed and safety, citing 73 and 65 published sources, respectively, concluded that "although evidence tended to support the notion that accident risk increased with speed, more study was needed to determine when changes in speed limit affect accidents or to predict the sizes of these effects."[43,44,45]

Although these issues can confuse the speed-safety paradigm, defined relationships among distance, time, and speed lead to the following six conclusions:

- The distance that a vehicle travels as a driver reacts to a particular scenario in the roadway ahead (e.g., lead vehicle slowing or stopping, pedestrian or bicyclist crossing traveled way) increases as speed increases.

- Braking distance increases as travel speed increases for a given deceleration rate.

- The time available to recover from a roadside encroachment decreases as speed increases.

- The probability that the forces required for a vehicle to maintain a circular path on a horizontal curve will exceed available forces (from friction and vehicle weight) increases as speed increases.

- The distance a vehicle travels while a driver is distracted (e.g., eye glances away from roadway ahead) increases as speed increases.

- The rate at which a driver must process information increases as speed increases.

These points indicate that, all else being equal, drivers traveling at faster speeds may be less likely to successfully react to unexpected situations (e.g., changes in lead vehicle behavior, nonmotorized users crossing traveled way, changes in roadway alignment, roadside encroachments) than drivers traveling at slower speeds.

The energy dissipated during a crash is directly proportional to the square of travel speed at the time of the crash. Impact forces affecting drivers increase as this initial speed increases and as the time over which energy is dissipated decreases. Therefore, the crash severity (i.e., probability of fatality or severe injury) increases as initial travel speed increases. Figure 10 depicts previous research (Joksch et al.) that demonstrates an exponential relationship.(46)

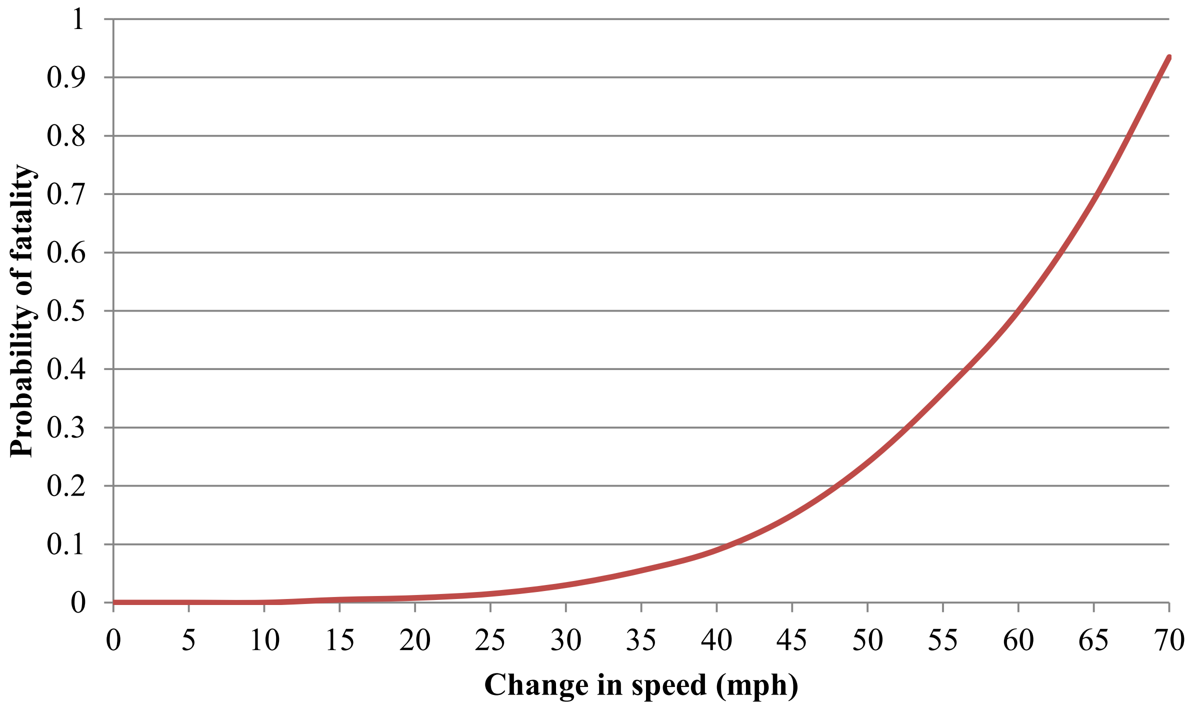

Figure 10. Graph. Relationship between change in travel speed during a crash and the probability of fatality (plotted model estimated by Joksch).(46)

The challenge to road designers, traffic engineers, and safety professionals is to design and retrofit roadways so that drivers choose speeds consistent with the intended operating speed of a facility (indirectly defined by the design speed, speed limit, and/or advisory speed).

Geometric Design Features

There are well-documented relationships among geometric design features, speed, and safety in the design consistency and crash prediction modeling literature for rural, two-lane highways. Similar literature on other rural and suburban roadway types is less developed. This section provides a general summary of the relationships among geometric design features, operating speeds, and safety on rural and suburban roadways. The discussion focuses on horizontal alignment, vertical grades, and cross-section elements. The research team reviewed domestic and international literature.

In much of the design consistency literature, researchers use statistical models to estimate the association between operating speeds and geometric design features. The most commonly modeled operating speed measures are the mean or 85th percentile speeds of a sample of drivers (mostly free-flow passenger cars, with free-flow defined as having a magnitude of a preceding headway greater than some value). The Operational Effects of Geometrics Committee (AHB65) of the Transportation Research Board (TRB) prepared the *Modeling Operating Speed Synthesis Report*, which is a combination of operating speed models developed in different regions of the world.[47] The research team used this report to prepare table 5, which summarizes modeled associations between design elements and operating speeds. The values shown are individual or range of estimated parameters from linear regression models. They represent the expected change in the magnitude and direction of operating speed associated with a unit change of the design element. For example, the lane width parameter of 2.08 indicates that operating speed is expected to decrease by 2.08 mph for every 1 ft reduction in lane widths. The parameter for the grade of horizontal curve approach tangent indicates that operating speed is expected to decrease by 1.47 mph for every 1 percent increase in grade.

Table 5. Statistical associations between vehicle operating speeds and geometric design features. (Information gathered from TRB Operational Effects of Geometrics Committee, 2011).[47]

	Change in Operating Speed per Unit Change in Geometric Feature (mph)		
Geometric Feature	Rural Two-Lane Highways	Rural Multi-Lane Highways	Suburban Highways
Lane width (ft)	0.032–2.08	N/A	N/A
Paved shoulder width (ft)	0.015–0.17	0.328	N/A
Length of horizontal curve approach tangent (ft)	-0.005–0.008	N/A	N/A
Grade of horizontal curve approach tangent (percent)	-1.47–3.11	N/A	N/A
Length of horizontal curve departure tangent (ft)	-0.002–0.023	N/A	N/A
Grade of horizontal curve departure tangent (percent)	-1.45– -0.224	N/A	N/A
Length of horizontal curve (ft)	-0.009–0.001	0.00002	N/A
Horizontal curve deflection angle (degrees)	-0.21– -0.08	N/A	N/A
Radius of horizontal curve (ft)	0.001–0.026	0.0005	N/A
Degree of horizontal curve	-0.481– -0.189	N/A	-17.90– -23.4
Rate of vertical curvature (ft/ft)	0.011	N/A	N/A

N/A = Not Available

Much of the work on statistical modeling of vehicle operating speeds has been for rural, two-lane highways. It appears that a unit decrease in the lane width is associated with a decrease in operating speed, although the magnitude of the decrease varies widely across published research. Similarly, a unit decrease in the paved shoulder width is consistently associated with a decrease in operating speeds. Narrowing lane and shoulder widths appears to be a possible speed-reduction measure on rural, two-lane highways. The research team also found consistency across two-lane rural highway speed prediction modeling research for the grade of the horizontal curve departure tangent, the horizontal curve deflection angle, and the degree of horizontal curve. All signs in table 5 for these variables are negative, which indicates that a unit increase in any of these dimensions is associated with a decrease in operating speeds. This suggests that increasing grades on the departure tangent, increasing the horizontal curve deflection angle, or increasing the degree of curve, will produce lower vehicle operating speeds. Note that the degree of horizontal curve is inversely related to the radius of curve, thus increasing the radius of curve is expected to increase operating speeds, which table 5 also illustrates. Porter et al. noted that the sensitivity of operating speeds to changes in geometry is relatively small:(22)

> The concept that road geometry can be adjusted to achieve consistency between target speeds and operating speeds is based on the premise that changes in geometry will affect operating speeds. Research has shown, however, that operating speeds may not be influenced by geometric design decisions unless very constrained dimensions are used. This may be counterintuitive given that geometric design criteria are dependent on design speed. (pp. 43–44)

There is limited published research relating geometric design features to vehicle operating speeds on rural, multi-lane highways and suburban roads. However, the statistical associations in table 5 for these two roadway types appear to confirm the findings for rural, two-lane highways—narrower paved shoulders and sharper horizontal curves are both associated with lower operating speeds, but the relationships are not highly sensitive.

Changes in road geometry associated with reductions in speeds (e.g., smaller horizontal curve radii, narrower lane widths, and narrower shoulder widths) are sometimes associated with increases in expected crash frequency. The following paragraphs discuss this phenomenon at greater length.

The *Highway Safety Manual* (HSM) contains crash modification factors (CMF) for geometric design features, particularly for rural, two-lane highways. Appendix A of this document includes available HSM CMFs for various geometric design features. In some cases, there are references to a figure or a table from the HSM. In other cases, an expected value is provided for the CMF with a standard error of the estimate.

Figure 11 shows an HSM sample of the lane width and shoulder width CMFs for rural, two-lane highways. In this case, the baseline condition is a 12-ft travel lane and a 6-ft shoulder, which has a combined CMF equal to 1.00 (i.e., the base condition). As illustrated in figure 11, it is expected that narrowing the lane width, shoulder width, or both will increase crash frequency. It is expected that CMFs related to other geometric elements, as well as to other facility types, follow similar patterns; narrower, geometries will result in higher crash frequencies.

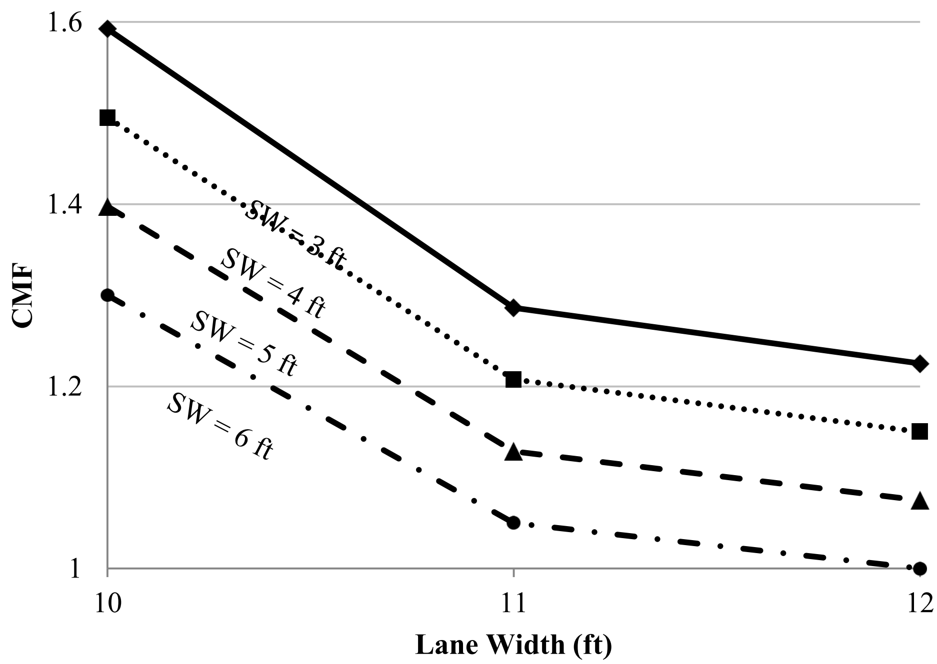

Figure 11. Graph. CMFs for combinations of lane and shoulder widths for two-lane rural highways (Average Annual Daily Traffic > 2,000 vehicles/day).

Gross et al. developed CMFs for lane-width–shoulder-width combinations on rural, two-lane highways; these are plotted in figure 12.[48] The interactions between lane width and shoulder width were accounted for during CMF estimation by using a case-control methodology. Results yield the following conclusions:

- Shoulder width has a larger effect on safety when lanes are narrow, but the effect of shoulder width decreases as lane width increases.

- An increase in lane width does not always result in an increase in safety, particularly when shoulder widths are wider.

- Rural, two-lane highway segments with lane-width–shoulder-width combinations totaling 16 to 17 ft (e.g., 10-ft lanes with 6-ft shoulders; 11-ft lanes with 6-ft shoulders, 12-ft lanes with 5-ft shoulders) may be safer than rural, two-lane highway segments meeting HSM base conditions (i.e., 12-ft lanes with 6 ft shoulders).

Estimated interactions between lane and shoulder width by Bonneson and Pratt have similar implications—the safety effect of lane width depends on the shoulder width. [49]

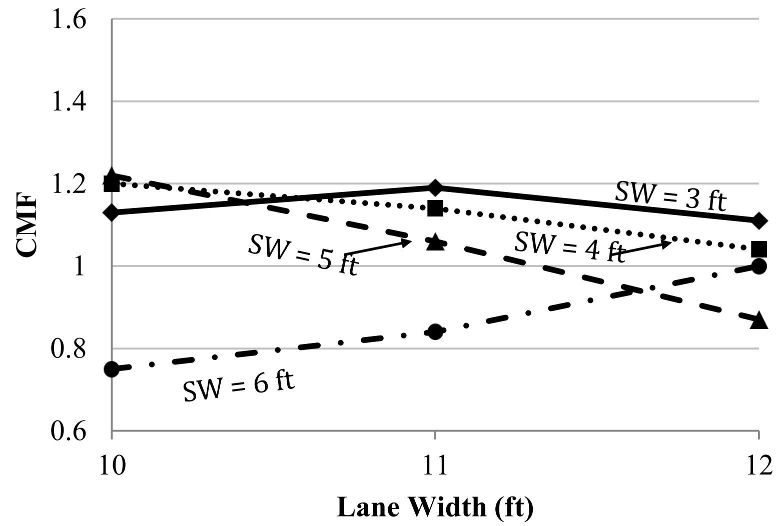

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Figure 12. Graph. CMFs for combinations of lane and shoulder widths for two-lane rural highways (figure 1 in Gross et al., 2009).[48]

Much of the research cited earlier in this section treats speed and safety effects of geometric design elements independently. Crash prediction algorithms currently in the HSM would lead geometric designers to conclude that larger radii horizontal curves, flatter vertical grades, and wider lane and shoulder widths will result in lower expected crash frequencies than geometric designs with smaller radius curves, narrower lane and shoulder widths, and steeper vertical grades. Speed prediction algorithms suggest that wider and flatter geometric designs produce higher vehicle operating speeds than do more curvilinear designs with narrower cross sections. Researchers explored the simultaneous effects of geometric design decisions on speed and safety by overlaying the results from the independently conducted speed and safety studies. Figure 13 through figure 15 provide the results.

The solid line in figure 13 represents the predicted 85th percentile speed for free-flow passenger cars as a function of horizontal curve radius on two-lane rural highways. The research team used models reported by Fitzpatrick et al. to create the operating speed line.[50] The dashed line represents the CMF for horizontal curve radius on two-lane rural highways from the HSM.[37] Horizontal curve radius clearly influences vehicle operating speeds; however, the effect is nominal until the radius falls below approximately 1,000 ft. Similarly, the expected crash frequency changes only nominally until the horizontal curve radius falls below 1,000 ft.

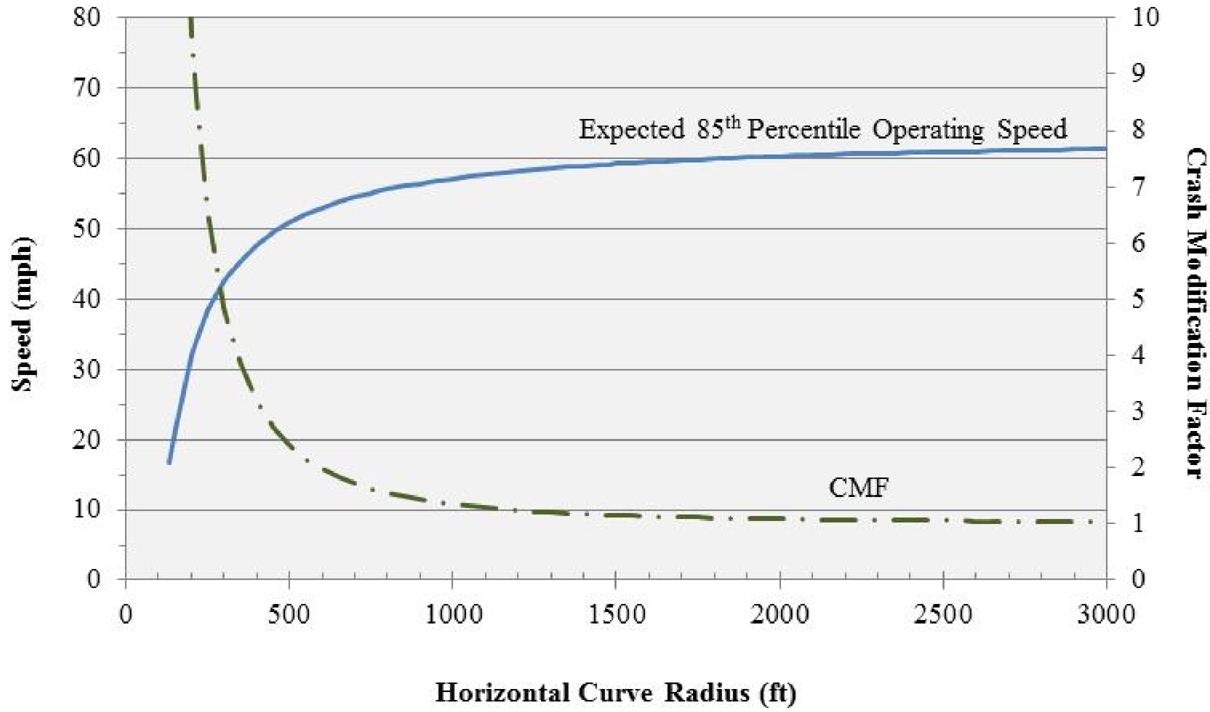


Figure 13. Graph. Relationships among horizontal curve radius, speed, and safety for two-lane rural highways.

Figure 14 shows the speed and safety effects for lane width on rural, two-lane highways. The solid line shows the predicted 85th percentile speed for free-flow passenger cars as a function of lane width on rural, two-lane highways. The research team used models reported by Lamm and Choueiri to create the operating speed line.(51) The dashed lines show the CMFs for lane width on rural, two-lane highways from the HSM for two levels of average daily traffic volumes.(37) The lane width is linearly related to the predicted operating speed—wider lane widths are associated with higher speeds. The expected crash frequency changes nominally for lane widths between 11 and 12 ft; however, the CMF increases significantly as lane widths are reduced below 11 ft. The lane width CMFs reported is applicable to run-off-road, sideswipe, and head-on collisions.

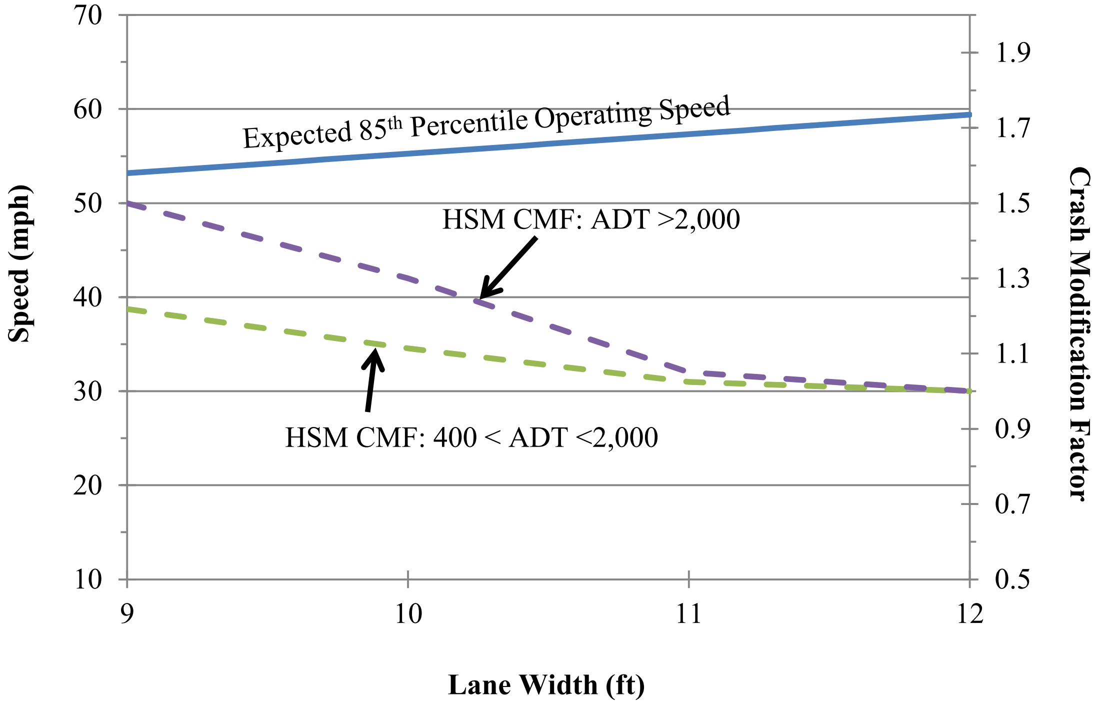


Figure 14. Graph. Relationship between lane width and speed and safety for two-lane rural highways.

Figure 15 shows the speed and safety effect tradeoff for the shoulder width on rural, two-lane highways. The solid line shows the predicted 85th percentile speed for free-flow passenger cars, as a function of shoulder width on rural, two-lane highways. Again, the research team used models reported by Lamm and Choueiri to create the operating speed line.(51) The dashed lines show the CMFs for shoulder width on rural, two-lane highways from the HSM for two levels of average daily traffic volumes.(37) The shoulder width is linearly related to the predicted operating speed—wider shoulder widths are associated with higher speeds. The expected crash frequency decreases in a linear manner as the shoulder width increases from 0 to 8 ft. As with lane width, the shoulder width CMFs reported are applicable to run-off-road, sideswipe, and head-on collisions.

The overall conclusion to be inferred from the speed and safety assessment of geometric design features is that more forgiving geometric designs generally tend to improve safety performance (i.e., fewer expected crashes), according to the HSM; however, more, smaller radii and narrower cross sections tend to produce lower vehicle operating speeds. The study by Gross et al. indicated that narrower cross-section geometries are not necessarily less safe and that the interactions between lane and shoulder widths are important to consider when evaluating safety effects (see figure 12, for example).(48) Because there was enough evidence to support exploring different combinations of cross-section elements (e.g., lane and shoulder widths) as a strategy to reduce operating speeds and improve safety on rural and suburban roads, FHWA recommended a new safety study on rural, two-lane roadway cross-section allocation. The study was conducted as part of the research project associated with this informational guide.

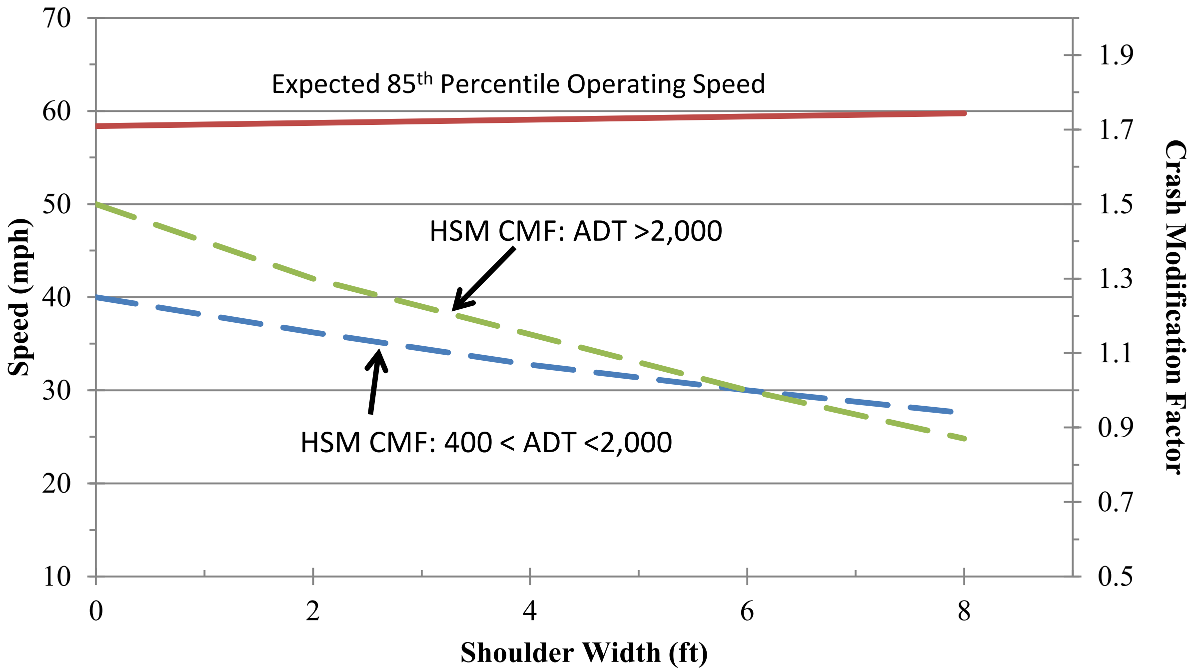


Figure 15. Graph. Relationship between shoulder width and speed and safety for rural, two-lane highways.

Engineering Treatments for Reducing Speeds

Traffic engineering treatments may affect speed or speeding-related crashes on rural or suburban roadways. Agencies sometimes implement speed-related traffic engineering treatments on or near a roadway section with the intention to modify driver operating speeds or reduce speeding-related crashes. The literature review identified more than 40 traffic engineering countermeasures that show promise of speed reductions or safety improvements on rural and suburban road segments. Appendix B includes a tabular summary of the countermeasures identified in both the international and domestic literature. This appendix also provides a description of the treatment, a photograph of the treatment (when available), the safety and operational effectiveness of each treatment, and cost information (if available). Readers are also referred to *Engineering Countermeasures for Reducing Speeds: A Desktop Reference of Potential Effectiveness*, *Speed Management: A Manual for Local Rural Road Owners*, and *Toolbox of Countermeasures for Rural Two-Lane Curves*.[5,6,52]

CHAPTER 3. GENERAL METHODOLOGIES AND SCOPE

The research team identified several treatments for possible field evaluations during this study and based the following list of possible treatments for field evaluations on findings of the literature review and State outreach efforts:

- **Converging chevron markings**: Published literature shows this treatment has reduced vehicle speeds by 2 to 3 mph on rural highways; however, there have been no documented safety evaluations to date.(53) This treatment can be applied using standard pavement marking materials and seems easy to maintain.

- **Narrower allocations of lane and shoulders widths on existing pavement**: Design consistency literature has shown that narrowing travel lanes or shoulders is associated with reduced speed.(48) Recent research suggests that reallocating lane and shoulder widths on an existing pavement can possibly improve safety.(53)

- **Speed tables**: Studies show this low-speed treatment can significantly reduce travel speeds on suburban streets, yet little is known about the safety effects of this treatment.(53,54)

- **Enhanced speed limit legend with colored surfacing**: The Iowa Department of Transportation documented that this treatment can significantly reduce vehicle operating speeds, but the sample of evaluation sites used is small and no safety evaluations have been documented to date.(53) This treatment can be applied using standard pavement marking materials and is seemingly easy to maintain.

- **Transverse markings or OSBs**: Published literature shows that this treatment has reduced vehicle speeds by 1 to 5 mph on rural; however, there are no documented safety evaluations to date. Highways. (See references 53 and 55 through 58.) This treatment can be applied using standard pavement marking materials and seems easy to maintain.

- **Red border speed limit sign:** Treatment has been shown to reduce vehicle operating speeds; however, there are no documented safety evaluations to date.(59) The costs to manufacture, install, and maintain this treatment are low.

- **HFST:** A single evaluation in Florida shows that this novel treatment can reduce vehicle operating speeds along curves; however, there are no robust safety evaluations reported to date.(60) There appear to be several candidate evaluation sites in four States. (Note: A before-after safety evaluation of this treatment is being completed under a separate FHWA project, "Evaluations of Low-Cost Safety Improvements Pooled Fund Study" (FHWA-HRT-14-065)).

- **Zigzag pavement markings:** This novel treatment has shown to reduce vehicle speeds; however, there are no safety evaluations reported to date.(61) This treatment can be applied using standard pavement marking materials and seems easy to maintain.

- **Speed feedback signs:** At least seven studies (not including those in school zones) show significant speed reductions result following speed feedback signs; however, the safety effects of these often short-term (less than 1 year) treatments is unknown. (See references 53 and 62 through 67.)

The FHWA technical review panel for this project organized these nine treatments into three groups: 1) treatments it has a high interest in evaluating, 2) treatments it has only a moderate interest in evaluating, and 3) treatments it has only a low interest in evaluating.

The panel's highest interest treatments were the following, in no particular order:

- HFST.
- Narrower allocations of lane/shoulder widths on existing pavement.
- Converging chevron markings.

Its moderate interest treatments were the following, in no particular order:

- Zigzag pavement markings.
- Transverse markings/OSBs.

The panel's low interest treatments were the following, in no particular order:

- Speed tables.
- Enhanced speed limit legend with color surfacing.
- Red border speed limit signs.
- Speed feedback signs.

The research team also assessed treatments based on State and local agency interests and their willingness to install them for this study purpose. For example, converging chevron pavement markings were not included in a treatment evaluation because of a lack of existing treatment locations on rural and suburban highways and the reluctance of agencies to install and maintain this treatment for the purposes of this study.

Based on feedback from FHWA, the current study completed the following evaluations:

- An observational before-after speed and driver behavior evaluation of the HFST on rural, two-lane horizontal curves. An observational before-after safety evaluation is being completed under a separate FHWA project, "Evaluations of Low-Cost Safety Improvements Pooled Fund Study (ELCSI–PFS)" (FHWA-HRT-14-065). Phase VI of this effort, Safety Performance Evaluation, includes the HFST safety evaluation.

- An observational before-after speed study of the OSB treatment on rural and suburban roads. In addition, the research team compiled all before-period crash data for the OSB treatment sites and comparable reference group sites into an analysis database and delivered to FHWA for future safety evaluations. At the time this final report was being prepared, less than 1 year had passed since the OSB treatments were deployed at most of the treatment sites. An adequate after-period was therefore not available for the safety evaluation. It is recommended that additional after-period crash data be collected during the years 2014 through 2016, and these data be added to the existing analysis database prior to completing the safety evaluation.

- A cross-sectional safety evaluation of the allocation of lane and shoulder width on existing rural, two-lane highways.

It should be noted that the FHWA selection of treatments to study as part of this effort is not necessarily correlated with magnitudes of expected speed reductions.

CRASH ANALYSIS FINDINGS SUMMARY

In addition to the operational and safety evaluations completed for the current study, the research team also examined State and local transportation agency police accident reports and completed a speeding-related, clinical crash analysis. The purpose of this assessment was to determine which speeding-related crash characteristics (e.g., driver age, gender, degree of familiarity with the crash location, weather conditions, and other factors) may influence speeding-related crash occurrence. This detailed analysis was intended to provide insights into crash causation that could not otherwise be determined by analyzing only electronically coded data. To perform this evaluation, the research team reviewed police reports from the following jurisdictions:

- Bedford, Shenandoah, and Stafford counties in Virginia.
- Huntingdon County in Pennsylvania.
- Van Buren County (Lawrence Village) and Grand Traverse County in Michigan.
- Hillsborough, Brevard, and Volusia counties in Florida.

The analysis considered speeding-related crashes that occurred in rural and small urban areas (population fewer than 50,000 persons), in the years 2004 to 2008. Most local transportation agencies that provided hardcopy speeding-related crash reports did not provide data concerning the total number of crashes occurring within the jurisdiction during the analysis period. Therefore, the proportion of the speeding-related crashes in the total crashes was not known.

The research team used the coding provided on the police accident report to identify speeding-related crashes. The following three driver actions, either singly or in combination, were identified as speed-related crashes:

- Speeding.
- Driving too fast for conditions.
- Failure to maintain proper speed.

When reviewing the hardcopy police accident reports, the research team not only assessed the coding provided by the investigating officer, but also reviewed the narrative and diagram of the crash location.

The research team identified a total of 1,895 nonintersection speeding-related crashes in the nine counties (four States) listed above; the team then excluded crashes with the following characteristics:

- Crashes involving drivers under the influence of alcohol or drugs.
- Collisions with animals.
- Crashes involving motorcyclists or tractor semi-trailers.
- Crashes involving roadway surface conditions that were icy, snow-covered, or with loose gravel.

These precipitating events in these crashes might not necessarily be attributable to speeding; consequently, driver impairment, poor weather conditions, differences in vehicle size and weight, and unexpected animal crossings were considered as factors not necessarily associated with speeding. After excluding these four crash types, 586 (30.9 percent) crashes met the speeding-related crash criteria described above.

The general findings from the clinical speeding-related crash analysis indicate the following:

- Nearly 62 percent of the 586 speeding-related crashes occurred on curves. While the mileage of horizontal curve and tangent alignment data within the nine counties in the analysis were not available for the current study, literature suggests that crashes on curves are three times as likely as crashes on tangents. Assuming that the mileage of curved roadway segments is lower than tangent roadway mileage in the data analysis files used in this study, the findings appear to support past research related to speeding-related crash occurrence.
    - Approximately 44 percent of all crashes on curves involved drivers younger than 21 years old, or with less than 3 years of driving experience.
    - A significant proportion (54 percent) of speeding-related curve crashes occurred on roadways that were likely familiar to the at-fault driver (within 10 mi of driver residence).
    - Almost 59 percent of all crashes on curves occurred on dry roadways with about 45 percent of crashes occurring during the daytime.
- About 38 percent of the analyzed speeding-related crashes occurred on tangents.
    - About 40 percent of all crashes involved drivers younger than 21 years or with less than 3 years of driving experience.
    - Nearly 69 percent of speeding-related crashes occurred on roadways that were likely familiar to the at-fault driver (within 10 mi of driver residence).
    - Almost 54 percent of all crashes occurred on dry roadways while about 73 percent occurred during daytime travel periods.

The findings from the clinical crash analysis suggest that inexperienced drivers are more likely to be involved in speeding-related crashes on curved alignments when compared with tangent alignments. In general, however, speeding-related crashes appear more likely on curved road segments when compared with tangent roadway segments. Speeding-related crashes on curves appear overrepresented at night when compared with daytime speeding-related crashes.

CHAPTER 4. OPERATIONAL EFFECTS OF HIGH-FRICTION SURFACE TREATMENT ON RURAL, TWO-LANE CURVES

Because a current observational before-after traffic safety evaluation is being performed for the HFST under another FHWA contract (Evaluation of Low-Cost Safety Improvements Pooled Fund Study), this study did not assess crash frequency or severity. Rather, the current study assesses the speed effects of HFST, as well as encroachments onto the shoulder or into the opposing travel lanes.

This section describes the HFST site selection criteria, site characteristics and data-collection, and analysis methods used for evaluation.

OVERVIEW

HFST can improve the skid-resistance properties of highway pavements. These surfaces are typically cold-applied treatments using a resin to bond polish- and abrasion-resistant aggregate to an existing pavement structure.[(68)] HFST has been applied in the United States on horizontal curves along roadway segments and on freeway interchange ramps; to delineate pedestrian, bicycle, and bus lanes; and on bridge decks. The roadway segment treatments are intended to reduce roadway departure crashes on horizontal curves and are applicable to the present study.

SITE SELECTION

Through outreach efforts to State transportation agencies, the research team identified several HFST horizontal curve treatment sites on two-lane rural roadways in West Virginia for use in this study. The West Virginia Department of Transportation (WVDOT) identified many high-risk locations on horizontal curves that were subject to run-off-road crashes and, in an effort to reduce injuries and fatalities, began implementing HFST in May 2011 as a low-cost, site-specific safety solution on the identified high-risk locations. The following roadways were selected for use in this study:

- West Virginia Route 32, mile marker (MM) 2.51, Randolph County.
- U.S. Route 33, MM 0.40, Pendleton County.
- U.S. Route 219, MM 5.81, Randolph County.
- U.S. Route 219, MM 6.32, Randolph County.

In addition to the four treatment sites, the research team identified three corresponding control sites. One of these control sites was located on the same route as two treatment sites and thus served as a control site for both treatment locations.

SITE CHARACTERISTICS

Table 6 shows the various site-specific data for all of the treatment and control sites. The research team selected treatment and control sites with the intent of collecting data at locations with varying horizontal curve radii. Control sites were located along the same route as the treatment sites. The research team tried to find control sites that were similar to the treatment sites with respect to horizontal curve direction, curve radius, curve length, superelevation, and PSL (including the advisory speed). The evaluation results section of this chapter provides a plan view sketch and details concerning each data-collection site, which show the data-collection locations with respect to the limits of each curve.

Table 6. West Virginia HFST site characteristics.

Site Number	County	Route	Radius of Curve (ft)	Length of Curve (ft)	Curve Direction	Vertical Grade (percent)	Superelevation (percent)	Posted/ Advisory Speed Limit (mph)
T1	Randolph	WV 32	680.4	1,080.5	Right	-0.5	8.0	55/40
C1	Randolph	WV 32	1073.3	1,213.6	Right	-3.5	7.0	55/50
T2	Pendleton	U.S. 33	210.0	539.2	Left	-8.0	6.0	55/25
C2	Pendleton	U.S. 33	264.8	693.6	Left	-7.5	8.5	55/25
T3	Randolph	U.S. 219 MM 6.32	605.1	753.8	Right	-2.0	11.0	55/30
C3/C4	Randolph	U.S. 219	544.6	826.9	Right	-6.5	12.0	55/30
T4	Randolph	U.S. 219 MM 5.81	272.9	673.4	Right	-3.0	12.0	55/25

MM = Mile Marker

The Before-Period Data Summary section later in this chapter describes the specific characteristics of each data-collection site.

INSTALLATION

Contractors for WVDOT installed HFST at the test sites between May 22, 2012, and June 4, 2012. HFST was applied by a self-contained, fully automated application vehicle in one pass up to 12 ft wide.

OPERATING SPEED EVALUATION METHODOLOGY

The research team employed observational before-after study to evaluate the effects of HFST on vehicle operating speeds. Because the intent is that HFST increases the available friction at the road surface–tire interface, this treatment may increase vehicle operating speeds. The basis for this hypothesis is that HFST improves the friction supply, and therefore, drivers may increase their travel speed and demand higher friction. Further, the HFST provides a color contrast when applied adjacent to older pavement surfaces and appears similar to a new pavement surface. This appearance may also encourage higher vehicle operating speeds. However, because an HFST typically is applied only between the limits of the horizontal curve (point of curvature (PC) to point of tangent (PT)), it is also possible that this treatment will have little influence on driver speed choice. An increase or no change in operating speeds will not necessarily imply a treatment failure. Comparisons of required versus available side friction for the speeds observed during this study, combined with the results of an ongoing FHWA safety evaluation of HFST, will indicate treatment effectiveness. The research team has also included an encroachment evaluation to supplement the speed evaluations and the parallel safety effort. Figure 16 and figure 17 show an example of a curve without the high-friction surface and the same curve with the high-friction surface, respectively.

Source: Pennsylvania State University

Figure 16. Photo. Photograph of a horizontal curve without the HFST.

Source: Pennsylvania State University

Figure 17. Photo. Photograph of a horizontal curve with the HFST.

The observational before-after evaluations were performed at four treatment sites, and three corresponding control sites. (One control site was located on the same route as two treatment sites and thus served as a control site for both treatment locations.) The control sites were located on the same roadway and had similar geometric features (i.e., lane width and curve radius) when compared with the treatment sites. The team collected data at each site for one before period and two after periods. This section of the report describes the data-collection equipment and locations, sample size requirements, data-collection periods and durations, and the statistical analysis methodology for the speed study.

Data-Collection Equipment and Locations

The research team used on-pavement traffic sensors to collect speed data. HI-STAR® sensors use vehicle magnetic-imaging technology to record vehicle count, speed, headway, time, pavement temperature, pavement condition (dry or wet), and vehicle length. These sensors are nonintrusive, thus reducing the possibility of drivers adjusting their speeds because of visible equipment and human observers. The dimensions of the sensors are 6.5 by 5.5 inches with a thickness of 0.625 inches. Each sensor is placed in the center of the travel lane and as a vehicle passes over it, the sensor captures changes in the magnetic field. A rubber cover protected the sensor and reduced its conspicuity on asphalt surfaces. The research team used Highway Data Management (HDM) software to program the sensors prior to field data collection—the internal clock was set to record vehicle speeds for up to 12 to 16 h, including both daytime and nighttime periods, depending on the time of day and traffic volumes present at the study locations. The research team also used HDM software to download the data from the sensors into Microsoft® Excel spreadsheets.

The research team placed the sensors at three different points at each study site to collect speed data at the treatment and comparison sites: 1) approximately 300 ft prior before the beginning of the horizontal curve,[1] 2) at the beginning of the horizontal curve (PC), and 3) at the midpoint of the horizontal curve. Figure 18 shows the data-collection locations along a study site. HI-STAR® sensors are capable of adding time stamp information to each vehicle collected at a particular location so vehicles can be tracked. A sensor was also placed in the opposing travel lane, near the midpoint of the horizontal curve, to determine whether a vehicle was present at the same time that a vehicle was present in the analysis direction.

The traffic volumes at the study locations were low, with daily traffic volumes ranging from approximately 700 to 1,600 vehicles per day. The probability of an opposing vehicle near to or on the horizontal curve at the same time as the subject vehicle was small. However, the analysis described below did consider the presence of a vehicle in the opposing travel lane.

[1]The research team selected the point speed location before the curve to capture speeds that were unaffected by the HFST. The “upstream” location of 300 ft before the beginning of the treatment was based on 3.65 s of preview time at 55 mph (81 ft/s).

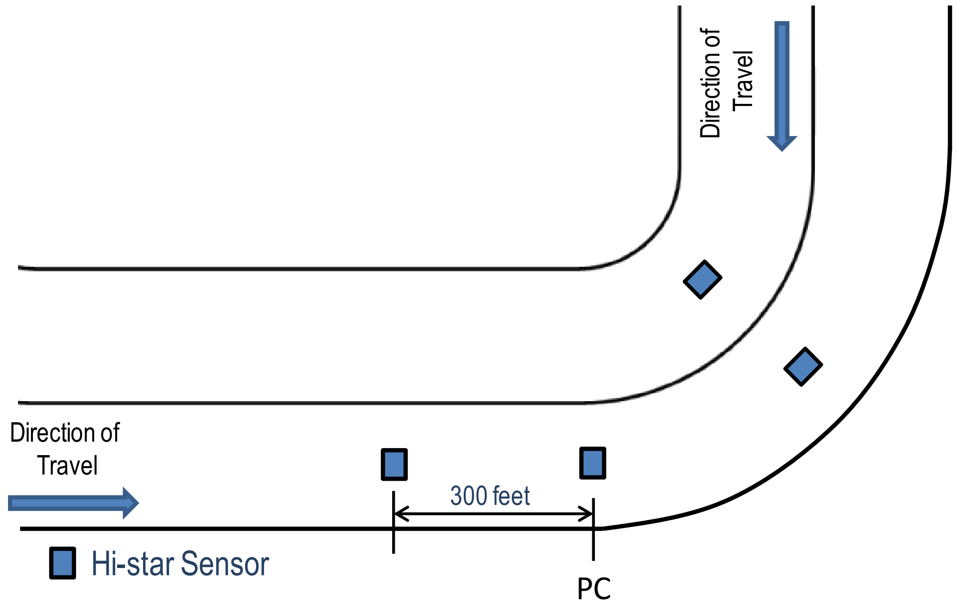


Figure 18. Diagram. Location of traffic sensors for treatment and control sites (not to scale).

Sample Size Determination

The research team used the equation shown in figure 19 to determine the sample size when estimating the mean speed from a population of speeds, while minimizing the tolerable error in field measurements.(69)

$$N = (S\frac{K}{E})^2$$

Figure 19. Equation. Minimum number of measured speeds.

Where:

N = minimum number of measured speeds.
S = estimated sample standard deviation (SD), mph.
K = constant corresponding to the desired confidence level.
E = permitted error in the average speed estimate, mph.

To obtain a range of possible sample sizes, multiple values for the confidence level K were considered. The values correspond to confidence levels of 90, 95, and 99 percent. The permitted error in the average speed estimate E has been included as a conservative value of ±1 mph. The estimate of sample SD S is a function of area type and highway type. The input value of 5.3 is representative of a rural, two-lane highway.(69) Table 7 summarizes the resulting sample size estimates for the proposed confidence interval, permitted error, and SD estimates.

Table 7. Sample size determination.

Confidence Level (percent)	K	N
90	1.64	76
95	1.96	108
99	2.58	187

As table 7 shows, the estimated sample size for the 95th percentile confidence level is approximately 110 free-flow passenger car speeds. It should be noted that collecting the minimum sample size depends on the traffic characteristics at each study site. The research team attempted to collect the minimum sample size during both daytime and nighttime periods, which covered one 12- to 16-h period per site (6−8 h of daytime and 6−8 h at night).

When comparing the difference in means for two different data-collection periods, the expected effect size (i.e., difference in mean speeds) is an important consideration. In this case, the equation shown in figure 20 was used to estimate the sample size in an observational before-after study:

$$N = \frac{4\sigma^2 \left(z_{critical} + z_{power}\right)^2}{D^2}$$

Figure 20. Equation. Sum of the sample in before-after comparison groups.

Where:

N = sum of the sample in both comparison groups (before-after).
s = SD of the comparison groups (assumed equal) (mph).
$z_{critical}$ = level of statistical significance assumed from the standard normal distribution.
z_{power} = desired statistical power from the standard normal distribution.
D = minimum expected difference between the sample means (mph).

Table 8 shows different sample size estimates, assuming that the SD in speed is 5.3 mph for varying levels of critical and power statistics and for different minimum expected differences in sample means.

Table 8. Sample size estimates for comparative research studies.

Difference in Means (mph)	Level of Statistical Significance ($z_{critical}$)			
	90 percent ($z_{critical}$ = 1.645)		**95 percent ($z_{critical}$ = 1.96)**	
	Desired Statistical Power (z_{power})		Desired Statistical Power (z_{power})	
	90 percent (z_{power} = 1.282)	95 percent (z_{power} = 1.645)	90 percent (z_{power} = 1.282)	95 percent (z_{power} = 1.645)
1.0	500	822	709	1,169
2.0	125	206	177	292
3.0	56	91	79	130
4.0	31	51	44	73
5.0	20	33	28	47

Because the treatments in the current study are original and little information exists to determine the expected difference in means between the before and after periods, the research team used

2.0 mph as a practical difference. In this case, 95-percent statistical significance and power required a minimum sample size of 292 free-flow passenger cars to be included in the combined before and after samples (i.e., 146 free-flow speeds per data-collection period). The 110 free-flow passenger car speeds recommended for each data-collection period based on the equation in Figure 19 would yield 95-percent statistical significance and more than 90-percent statistical power, if a 2.0 mph difference in means was computed between the before and after samples. As a result, the research team sought to measure at least 110 free-flow vehicle speeds during each 12- to 16-h collection period; however, if additional free-flow vehicles were available for analysis, the entire sample was used to compare the before with after mean speeds.

Data-Collection Periods and Durations

Speed data were collected on weekdays only and during both daytime and nighttime periods. Only free-flow vehicles were included in the evaluation sample. The definition of a free-flow vehicle also meant no opposing vehicles were present that could influence the subject vehicle. The extent to which observations with opposing vehicles could be screened depended on the sample size and the estimated effect of the opposing vehicles as described above. Based on previous studies, free-flow vehicles were identified as those vehicles traveling with a minimum headway of 5 s. (70,71) Data were collected at each study site during favorable driving conditions—clear weather with normal visibility (no fog) and dry roadway conditions, without the presence of standing water from an earlier rain. These requirements increased the likelihood that drivers were selecting their operating speeds based solely on the physical roadway and roadside features at each site; during and after HFST implementation, any changes in operating speeds could then be associated with the HFST treatment. The data-collection session was long enough so that sufficient data were collected to satisfy the sample size requirements after the data were screened (i.e., after non-free-flow passenger cars were eliminated from the database).

The study methodology was an observational before-after study. The before data were collected before implementing the HFST on horizontal curves. All before data collection occurred during summer 2012. Two after data collection periods were included in the evaluation. The first after period occurred approximately 1 month after implementing the HFST to minimize the novelty effects associated with the treatment—these data were also collected during summer 2012. A second after period occurred approximately 1 year after implementing the HFST, during summer 2013. The research team chose a 1-year interval between the after periods to minimize seasonal effects and to assess long-term novelty effects of the treatment. It was anticipated that the skid-resistant qualities of the HFST would decrease after 1 year because of vehicle tire abrasion and likely winter maintenance on roadways in West Virginia.

Statistical Analysis

Before beginning the data analysis, all raw data from the treatment and control sites were screened to exclude all vehicles that were not passenger cars and that were not considered free-flow vehicles, as described previously. It was anticipated that vehicles present in the opposing travel lane would influence the speed of free-flow vehicles in the subject travel lane; therefore, the research team added an indicator variable to the data files to identify free-flow vehicles in the subject travel lane that were measured in the presence of an opposing vehicle. The research team excluded missing data values from the analysis. The research team carefully evaluated data points considered outliers to determine whether to exclude them in the analysis. Examples of outliers include vehicles traveling at very low speeds or entering or exiting at nearby driveways.

The research team calculated after data screening and mean operating speeds at each sensor location for all treatment and control sites and all data-collection periods. The team then compared corresponding speed parameters at each site for each data-collection period by calculating the numerical differences in these speed parameters. The team applied the *t*-statistic for independent samples to determine whether the differences in mean speed were statistically significant. Statistically significant changes in mean operating speeds indicated that the observed speeds were different in the two time periods compared. The *t*-statistic is commonly used to test the hypothesis of differences in population parameters.[(72)] In this study, the null and alternative hypotheses for testing the differences in two population mean speed measures, μ_1 and μ_2, were the following:

- Null Hypothesis (H_0): There has not been a change in mean speeds as a result of HFST implementation, or H_0: $\mu_1 - \mu_2 = 0$.

- Alternative Hypothesis (H_a): There has been a decrease in mean speeds as a result of HFST implementation, or H_a: $\mu_1 - \mu_2 > 0$.

At each study site, a *t*-statistic is calculated for each sensor location and between data-collection periods. Independent two-sample *t*-statistics is applied to test for the difference between two sample means at each study site. The *t*-statistic for large samples with known variables is calculated using the equation shown in figure 21: [(73)]

$$t = \frac{(\overline{X}_B - \overline{X}_A)}{\sqrt{\frac{s_B^2}{n_B} + \frac{s_A^2}{n_A}}}$$

Figure 21. Equation. *t*-statistic to test for the difference between two sample means at each study site.

Where:

$\overline{X}_B, \overline{X}_A$ = mean speed for the before and after periods.
s_B, s_A = SD of speed for the before and after periods.
n_B, n_A = sample size in before and after periods.

In figure 21, both the first and second after periods were compared with the before period and to each other, using data from the treatment and comparison sites.

The degrees of freedom (df) for the independent samples *t*-statistic is $n_A + n_B - 2$. The critical value when $\alpha = 0.05$ for a one-tail test is 1.645. The null hypothesis is rejected when the computed *t*-test exceeds the critical value, thus concluding that the mean speeds being compared differ between the two collection periods being considered. An alternative method to determine the statistical significance of HFST on mean speed is the *p*-value associated with the *t*-statistic. A low p-value (i.e., less than or equal to 0.05) indicates a high probability that implementing the HFST influenced mean speeds between two data-collection periods. The team computed t-statistic and p-value for each pair of collection periods at each study site. It was anticipated that the difference in mean speeds at the control sites would not be statistically significant. However, if there is a statistically significant difference in mean speeds between any of the data-collection periods, the magnitude of this difference would need to be taken into account. In this case, the research team added or subtracted, depending on whether the difference is positive or negative, to the numerator in figure 21. As such, the mean speed difference computed at the treatment sites was adjusted to account for statistically significant mean speed differences at the treatment sites.

The research team also used the t-test for independent samples to compare the before and after mean difference in vehicle speeds between the PC and the curve midpoint. The comparison of mean delta v, where delta v is the curve PC speed minus the curve midpoint speed, was made in the same way as the mean speed comparisons.

In addition to the t-test, the percentage of vehicles exceeding the PSL and the advisory speed at the treatment and control sites was calculated and compared between data-collection periods. The percentage of speeding vehicles, P_S, was computed using the equation shown in figure 22.

$$P_S = \frac{x}{n} \times 100$$

Figure 22. Equation. Percentage of speeding vehicles.

Where:

x = number of vehicles exceeding the PSL.
n = the total number of vehicles in the sample.

By comparing the number of vehicles exceeding the PSL in two data-collection periods, it can be determined whether the HFST is associated with reducing the proportion of PSL violations. The percent reduction in speeding vehicles $\%R_S$ from period 1 to period 2, at the treatment and control locations, is computed using the equation shown in figure 23.

$$\%R_S = \frac{P_{SB} - P_{SA}}{P_{SB}} \times 100$$

Figure 23. Equation. Percent reduction of speeding vehicles.

Where:

P_{SB} = the proportion of vehicles speeding during the before data-collection period.

P_{SA} = the proportion of vehicles speeding during the after data-collection period. (Note that there will be two after periods for each location.)

To determine whether the proportion of vehicles exceeding the PSL at the treatment and control locations changed between data-collection periods, a *Z*-test for independent samples was computed. The null and alternative hypotheses for the test are the following:

- Null Hypothesis (H_0): There is no difference between the two sample proportions, or H_0: $P_{SB} - P_{SA} = 0$.
- Alternative Hypothesis (H_a): There is a difference between the two sample proportions, H_a: $P_{SB} - P_{SA} \neq 0$.

The *Z*-statistic used to determine the statistical difference between the two proportions is computed using the equation shown in figure 24.(73)

$$Z = \frac{P_{SB} - P_{SA}}{\sqrt{P(1-P)\left(\frac{1}{n_1} + \frac{1}{n_2}\right)}}$$

Figure 24. Equation. *Z*-statistic to determine the speed difference between a pair of before and after periods.

Where P_{SB} and P_{SA} are the sample proportions from figure 23, n_1 and n_2 are sample sizes for the corresponding proportions being considered, and P is the combined proportion in both samples, computed using the equation shown in figure 25.

$$P = \frac{x_B + x_A}{n_B + n_A}$$

Figure 25. Equation. Combined proportion of vehicles speeding during the before and after data-collection periods.

Similar to the *t*-statistic, the *Z*-statistic is associated with a *p*-value. A *p*-value of 0.05 or less rejects the null hypothesis and concludes that the HFST is effective in reducing the number of vehicles exceeding the speed limit at the 95-percent confidence level.

The final speed performance metric considered in the present study is speed variance. A two-sided *F*-test is used to compare the variances of vehicle operating speed in the before and after periods. The *F*-test is computed using the equation shown in figure 26.

$$F = \frac{s_B^2}{s_A^2}$$

Figure 26. Equation. F-test to compare the speed variance in the before and after periods.

This equation has an *F*-distribution with $(n_B - 1)$ numerator degrees of freedom and $(n_A - 1)$ denominator degrees of freedom. When the computed *F*-test exceeds the critical value, the null hypothesis is rejected, and the conclusion is that the speed variance differs between the before and after periods.

HFST ENCROACHMENT EVALUATION METHODOLOGY

The purpose of the encroachment evaluation was to determine whether the HFST changes the proportion of vehicles that cross either the edge line or centerline on two-lane rural highways in West Virginia. Because the HFST increases the coefficient of road adhesion, it was hypothesized that the proportion of lane-line encroachments would decrease. The following sections describe the encroachment evaluation data collection and analysis.

Data-Collection Locations and Periods

Data collection took place at the same locations described above in the speed evaluation section. To identify lane-line encroachments, the research team deployed digital video recorders at all treatment and comparison site locations. A single recorder was positioned on the approach tangent, near the PC location, to record vehicles traversing the first half of the horizontal curve (PC to midpoint). Video data were recorded during daylight hours only in the before period (2 to 4 h per collection period at each site), and during two after treatment periods. The first after period occurred within the first month after applying the HFST to a horizontal curve, and the second after period occurred approximately 1 year after installing the HFST treatment. Data collection at the comparison sites occurred on the same days as the treatment site data collection.

Analysis Methods

The research team collected the video data and reviewed it on a computer. The research team then tabulated the total number of vehicles observed and the number of vehicles that crossed the edge line and the centerline of the roadway within the limits of the PC and midpoint of the horizontal curve. The proportion of vehicles encroaching on the edge line or centerline was then computed using the equation shown in figure 27.

$$P_e = \frac{x}{n} \times 100$$

Figure 27. Equation Proportion of vehicles encroaching on the edge line or centerline.

Where:

P_e = proportion of vehicles encroaching on the edge line or centerline.
x = number of vehicles encroaching on the edge line or centerline.
n = number of vehicles observed during data-collection period.

To determine whether the proportion of vehicles encroaching on a lane line at the treatment and control locations changed between the before and after data-collection periods, the team computed a *Z*-test for independent samples. The null and alternative hypotheses for the test are the following:

- Null Hypothesis (H_0): There is no difference between the two sample proportions, or H_0: $P_{e(\text{before})} - P_{e(\text{after})} = 0$

- Alternative Hypothesis (H_a): There is a difference between the two sample proportions, H_a: $P_{e(\text{before})} - P_{e(\text{after})} \neq 0$.

The *Z*-statistic used to determine the statistical difference between the two proportions is shown in figure 28.

$$Z = \frac{P_{e(before)} - P_{e(after)}}{\sqrt{P(1-P)\left(\frac{1}{n_1} + \frac{1}{n_2}\right)}}$$

Figure 28. Equation. *Z*-statistic to determine the proportion of vehicles encroaching on a lane line during the before and after data-collection periods.

Where $P_{e(before)}$ and $P_{e(after)}$ are the sample proportions from figure 23, n_1 and n_2 are sample sizes for the corresponding proportions being considered, and P is the combined proportion in both samples, computed using the equation in figure 29.

$$P = \frac{x_{before} + x_{after}}{n_{before} + n_{after}}$$

Figure 29. Equation. Combined proportion of vehicles encroaching on a lane line between the before and after data-collection periods.

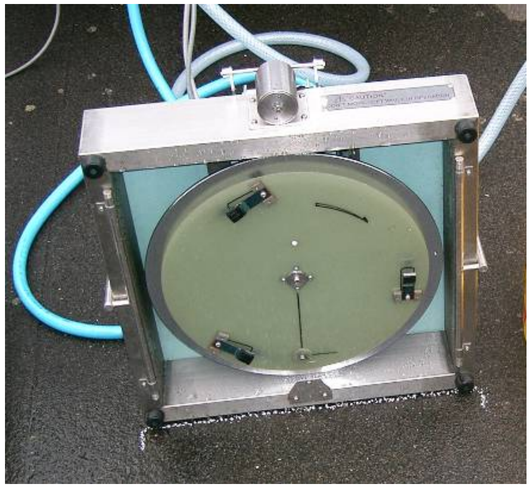
Similar to the *t*-statistic, the *Z*-statistic is associated with a *p*-value. A *p*-value of 0.05 or less rejected the null hypothesis and concluded that the HFST is effective in reducing the number of vehicles exceeding the speed limit at the 95-percent confidence level.

FRICTION EVALUATION METHODOLOGY

The research team also measured roadway surface friction before and after implementing HFST. The following is a discussion of the friction testing methods.

The research team used a dynamic friction (DF) tester to evaluate the skid resistance of the HFST and comparison site locations. All testing was performed in accordance with ASTM E1911-09a, *Standard Test Method for Measuring Pavement Surface Frictional Properties Using the Dynamic Friction Tester*. The DF tester measures the necessary torque to turn three small rubber pads in a circular path on the measured surface at different speeds. The apparatus consists of a horizontal spinning disk fitted with three spring-loaded rubber sliders that contact the paved surface as the disk rotational speed decreases because of the friction supplied between the sliders and the paved surface. A water supply unit delivers water to the paved surface being tested. The torque generated by the slider forces, measured during the spin down, is then used to calculate the friction supply as a function of speed. Typical test speeds range from 55 mph down to 3 mph. The device uses an electric motor to spin the measuring disc at the desired speed and an electromagnetic device to lower the spinning disk to the ground at the highest speed revolution.

The device is equipped with a rotational speed measurement device together with a rotational torque and a downward load measurement sensor. Figure 30 shows the DF tester.



Source: Pennsylvania State University

Figure 30. Photo. Dynamic friction tester.

The device is portable and was manually placed on the pavement surface where the test took place. The research team used a laptop computer to control the test and record the data. Once the test was initiated, the electronic motor first accelerated the disk to the standard spinning speed of 55 mph. The electromagnetic release mechanism then dropped the spinning disk to the ground, at which time automated data acquisition began. The test was completed when the disk came to a complete stop. The raw data were then filtered—the friction supply at the pavement-rubber disk interface was calculated from the measured and filtered torque and loading forces based on an internal algorithm within the testing apparatus.

The team used a circular texture (CT) meter along with the DF tester to measure road surface texture characteristics. The meter is designed to measure surface texture on the same circular track as the DF tester. The CT meter calculates and reports the mean profile depth (MPD) of the road surface and the International Friction Index (IFI).

The following describes the data-collection protocol with the number of measurements and the placement of measurements.

Data-Collection Protocol

The research team used the DF tester and CT meter to measure all treatment and control sites included in the speed study. A single before period, and two after periods were included in the data-collection process. These periods corresponded with the time periods described previously for the speed and encroachment data collection. The research team used the following testing protocol:

1. Each test section was divided into two segments as follows:

    - The first segment was the last 300 ft (if available) of the approach tangent before the beginning of a horizontal curve.
    - The second segment was the entire length of the treated horizontal curve.

2. The first segment was divided into three equal 100-ft sectors. The research team used the DF tester and CT meter devices to measure the beginning and end point of these sectors. This yielded four total friction supply measurements on the approach tangent before the horizontal curve. It was assumed that this segment is where deceleration was most likely to occur and thus the texture and friction supply of the pavement surface was likely to vary on the approach tangent. The proposed measurements enabled the research team to determine this variability in friction supply. All friction supply measurements on the approach tangent were taken in the left wheel path.

3. The second segment (i.e., horizontal curve) was similarly divided into three equal length sectors yielding four physical measurement locations. The intent of measuring friction supply at four locations within the horizontal curve was to provide information about the variability in friction supply within the limits of the curve.

4. Each measurement location within a sector was defined as a 6-ft-long straight line. The research team again used the CT meter and DF tester devices to measure the beginning and end points of the 6-ft-long line, producing two individual measurement points for each location. The research team computed the average of these measurements, which served as the measured data for the locations.

5. Within each horizontal curve segment, the friction supply measurement locations were determined as follows:

    - On curves to the right, all measurements were recorded in the left wheel path because this location experiences more polishing and therefore supplies less friction than the right wheel path.
    - On curves to the left, friction supply was measured in the right wheel path.

Analysis

The research team derived a representative friction supply curve for each test segment. The curves were plotted to illustrate the difference in friction supply before and after application of the HFST. Figure 31 shows a sample illustration of road friction curves produced using the measurement protocol described above. The research team used this analysis to determine the absolute change in friction supply at various locations approaching and along the horizontal curve at the treatment and control sites.

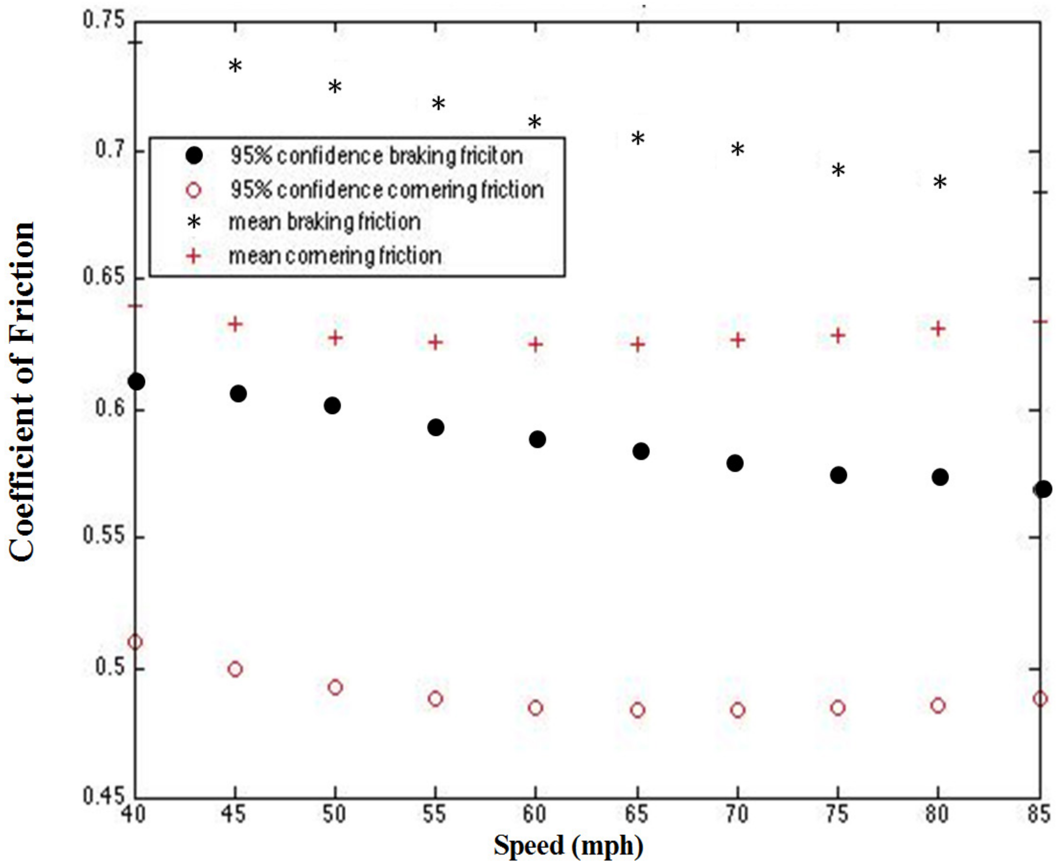


Figure 31. Graph. Road friction curves (for illustrative purposes only).

In addition to the friction curves, the research team used the speed data collected during the speed study to compute the difference in friction supply and the friction demanded by vehicles traversing horizontal curves at the treatment and control locations. The friction demand was computed using the point-mass model, using the equation shown in figure 32.

$$f = \frac{V^2}{15R} - \frac{e}{100}$$

Figure 32. Equation. Side friction demand.

Where:

f = side friction demand.
V = vehicle operating speed (mph).
R = radius of horizontal curve (ft);
e = rate of superelevation (percent).

During the friction testing, the research team used a digital slope meter to measure the superelevation at the midpoint of each horizontal curve. The team then computed the descriptive statistics from the distributions of friction demand (e.g., mean, SD, 95th percentile) at each treatment and control curve location to the friction supply distribution. The margin of safety, based on the difference between the distributions of friction supply and demand, was computed for each treatment and control location.

BEFORE-PERIOD DATA SUMMARY

Speed, friction, and encroachment data were collected before implementing the HFST application at seven sites near Elkins, WV. Data were collected at four treatment sites and three comparison sites. One comparison site was located between two treatment sites on U.S. Route 219, and was therefore used as a basis to compare the before and after operational performance at the two adjacent treatment sites. This section of the report describes the specific characteristics of each data-collection site, as well as the speed and other operational data-collection protocols employed at each site in the before period. This section of the report also provides results of the before analysis.

West Virginia Route 32 Treatment Site (T1 in Table 6)

The direction of travel for speed data collection was from south to north (bottom left to top right in figure 33). There was a cut slope on the inside of the curve, which limited horizontal sight distance along the curve. The radius of curve and the curve length, calculated using Google Earth™, were found to be 680.43 ft and 1,080 ft, respectively. The superelevation for this curve was 8.0 percent and the vertical grade was -0.5 percent, based on manual field measurements with a digital slope meter. The PSL was 55 mph. As figure 33 shows, there was a curve-to-the-right warning sign (W1-2) with a 40-mph advisory speed plaque (W13-1P). The travel lanes were 10 ft wide, and there was a 4-ft paved shoulder on both sides

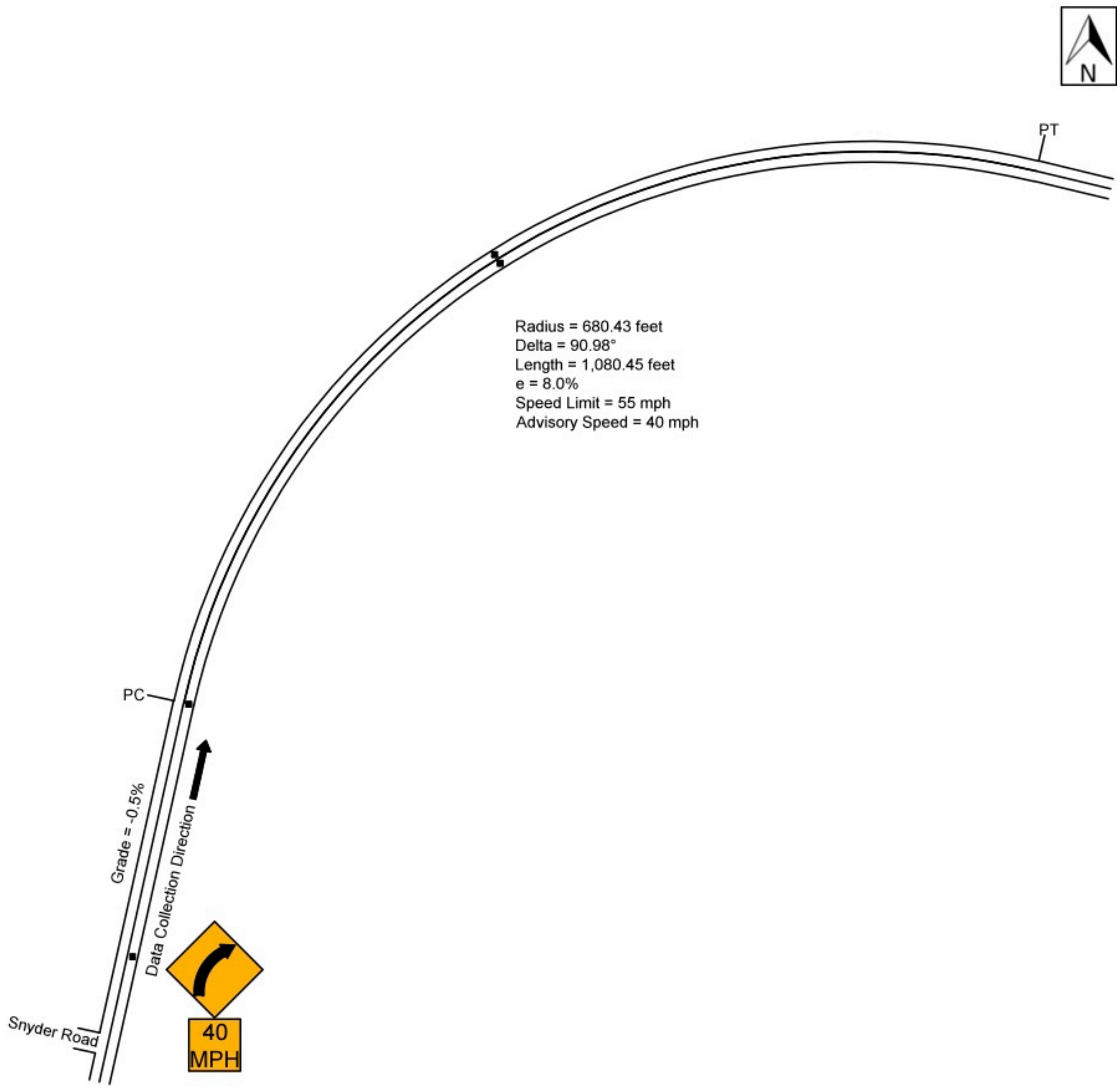


Figure 33. Diagram. Geometric layout of WV Route 32 treatment site (not to scale).

At this site, the research team collected data 300 ft before the PC, at the PC, and at the midpoint of the horizontal curve. A sensor was placed at the midcurve location in the opposing direction of travel to determine whether an opposing vehicle was present during data collection in the northbound travel lane. A camera was placed approximately at the PC to collect braking and encroachment data for 2 h during daylight. Braking data were only collected for the northbound traffic, while encroachment data were collected in both directions of travel. An intersection (WV Route 32/4) was located approximately 200 ft before the first sensor in the northbound direction of travel. Figure 33 shows a graphical layout of the horizontal curve along with the speed data-collection locations at the WV Route 32 treatment site.

Table 9 shows the speed data collected during the before period. The data are presented separately for passenger cars and heavy trucks, daytime and nighttime speeds for passenger cars, opposed (a vehicle present in the opposing lane) versus unopposed passenger cars, and free-flow versus non-free-flow passenger cars. All of these metrics are presented for the approach, PC, and midpoint of the horizontal curve locations. An independent sample *t*-test was performed to compare the mean speed and speed deviation for the disaggregate data comparisons shown in table 9. At the WV Route 32 treatment site, there was a statistically significant difference in mean speeds at the PC for passenger cars versus heavy vehicles and for opposed versus unopposed vehicles. As table 9 shows, there were only 9 heavy vehicles versus 289 passenger cars. There were 44 opposed vehicles versus 245 unopposed vehicles in the data-collection period. All other statistical comparisons were not statistically significant at the 95th percentile level ($\alpha = 0.05$). This includes the comparison of daytime versus nighttime free-flow vehicle speeds and the comparison of free-flow versus non-free-flow vehicle speeds. The latter finding was expected because the traffic volumes were low at this treatment site.

Table 9. WV Route 32 treatment site before data.

Speed Metric	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N
Passenger cars	54.796	6.698	**53.052**	6.174	47.803	5.493	289
Heavy trucks	55.111	3.983	**51.111**	4.781	47.556	5.175	9
Daytime passenger cars	54.897	6.809	53.231	6.209	47.872	5.474	234
Nighttime passenger cars	54.364	6.246	52.291	6.021	47.509	5.614	55
Opposed passenger cars	53.886	5.735	**50.614**	6.300	47.455	5.214	44
Unopposed passenger cars	54.959	6.854	**53.490**	6.061	47.865	5.550	245
Free-flow passenger cars	54.976	6.955	53.172	6.316	47.948	5.557	250
Non-free-flow passenger cars	53.641	4.637	52.282	5.176	46.872	5.027	39

Bold indicates statistical significance at the 95-percent confidence level (α=0.05).
SD = Standard Deviation
PC = Point of Curvature
N = Number of Observations

Figure 34 graphically shows the PSL, advisory speed limit, and the mean speeds for passenger cars and trucks along the WV Route 32 treatment section during the before period. As the figure shows, the truck and passenger car mean speeds are similar across the study section, but the heavy trucks appear to have a more constant deceleration from the approach to the midpoint of the curve. Passenger cars appear to maintain speed from the approach to the PC and then decelerate once into the curve. The mean acceleration rate from the PC to the midpoint of the curve was -1.15 ft/s (with a SD of 1.256) for passenger cars and -0.609 ft/s (0.833) for heavy trucks. A negative value indicates deceleration.

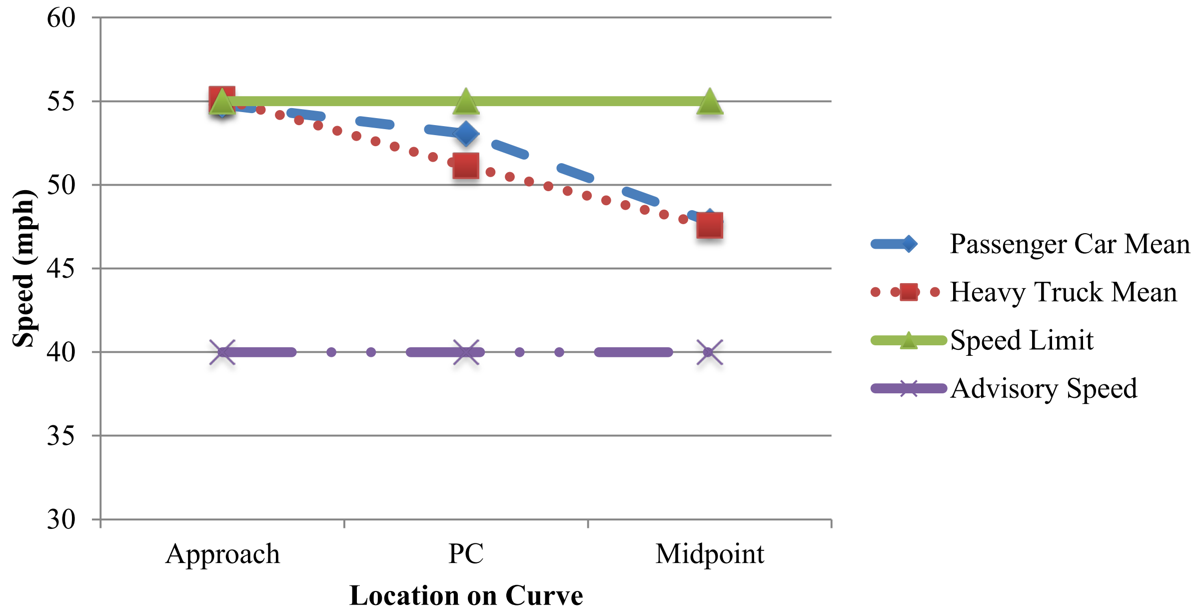


Figure 34. Graph. Graphical representation of speeds at the WV Route 32 treatment site.

Friction data were collected at eight locations at this study site, as described in the Friction Evaluation Methodology section. Table 10 shows the data-collection locations and the measurements that were taken for pavement macro-texture and pavement micro-texture. The pavement macro-texture was measured using the CT meter in accordance with ASTM E2517 to establish the MPD in millimeters. The frictional micro-texture characteristic of the sample surfaces was measured with the DF tester device according to the ASTM E1911 standard. The two complementary devices allowed the research team to calculate the IFI friction indices and subsequently translate the IFI numbers into predicted locked wheel friction numbers (Skid Number (SN) 65). Table 10 shows the SN 65 values based on the macro- and micro-texture measurements.

Table 10. WV Route 32 treatment site friction data.

Location	PC +300	PC + 200	PC + 100	PC	1/4	Mid	3/4	PT
CTM MPD	0.750	0.790	0.943	0.977	0.930	0.987	0.940	0.980
DFT 20	0.530	0.595	0.495	0.490	0.395	0.480	0.385	0.400
SN 65	0.410	0.460	0.400	0.390	0.320	0.380	0.310	0.320

CTM = Circular Texture Meter
MPD = Mean Profile Depth
DFT = Dynamic Friction Meter
SN = Skid Number
PC = Point of Curvature
PT = Point of Tangent

As table 10 shows, the skid number at 40 mph is lower within the curve than before the curve, which was expected as a result of greater friction demand in the lateral direction when cornering.

The research team collected encroachment data for 127 vehicles in the northbound direction and 132 vehicles in the southbound direction. Seventeen vehicles encroached onto the shoulder, and no vehicles crossed the centerline of the road in the northbound direction. Twelve vehicles crossed the centerline and 6 vehicles encroached onto the shoulder in the southbound direction. Braking maneuvers were also observed for the northbound vehicles. Because the camera was located at the PC, only vehicles braking within the curve were observed. At this site, 35 of the 127 vehicles were observed to brake within the curve for the northbound direction.

West Virginia Route 32 Comparison Site (C1 in Table 6)

Data were also collected at a comparison site approximately 1 mi south of the treatment site on WV Route 32 at the same time of the same day as the treatment site. The direction of travel was also south to north on a similar horizontal curve to the right. The radius of curve and the curve length, calculated using Google Earth™, were found to be 1,073.3 ft and 1,213.6 ft, respectively. The superelevation for this curve was 7.0 percent, and the vertical grade was -3.5 percent, based on manual field measurements with a digital slope meter. There was a small slope on the inside of the curve that limited horizontal sight distance along the curve, and there was a driveway inside the curve, approximately at the quarter-point between the PC and the PT. The PSL was 55 mph. As shown in figure 35, there was a reverse curve warning sign (W1-4) with a 50-mph advisory speed plaque (W13-1P). The travel lanes were 10 ft wide, and there was a 4-ft shoulder on both sides.

Speed data were collected 300 ft before the PC, at the PC, and at the midpoint of the curve. The research team placed the sensor in the opposing direction of travel at the midpoint of the curve to determine whether vehicles were present in the opposing lane. The camera was placed upstream of the PC (approximately 100 ft) to observe braking before and into the curve for northbound vehicles. Figure 35 shows a graphical layout of the horizontal curve along with the speed data-collection locations.

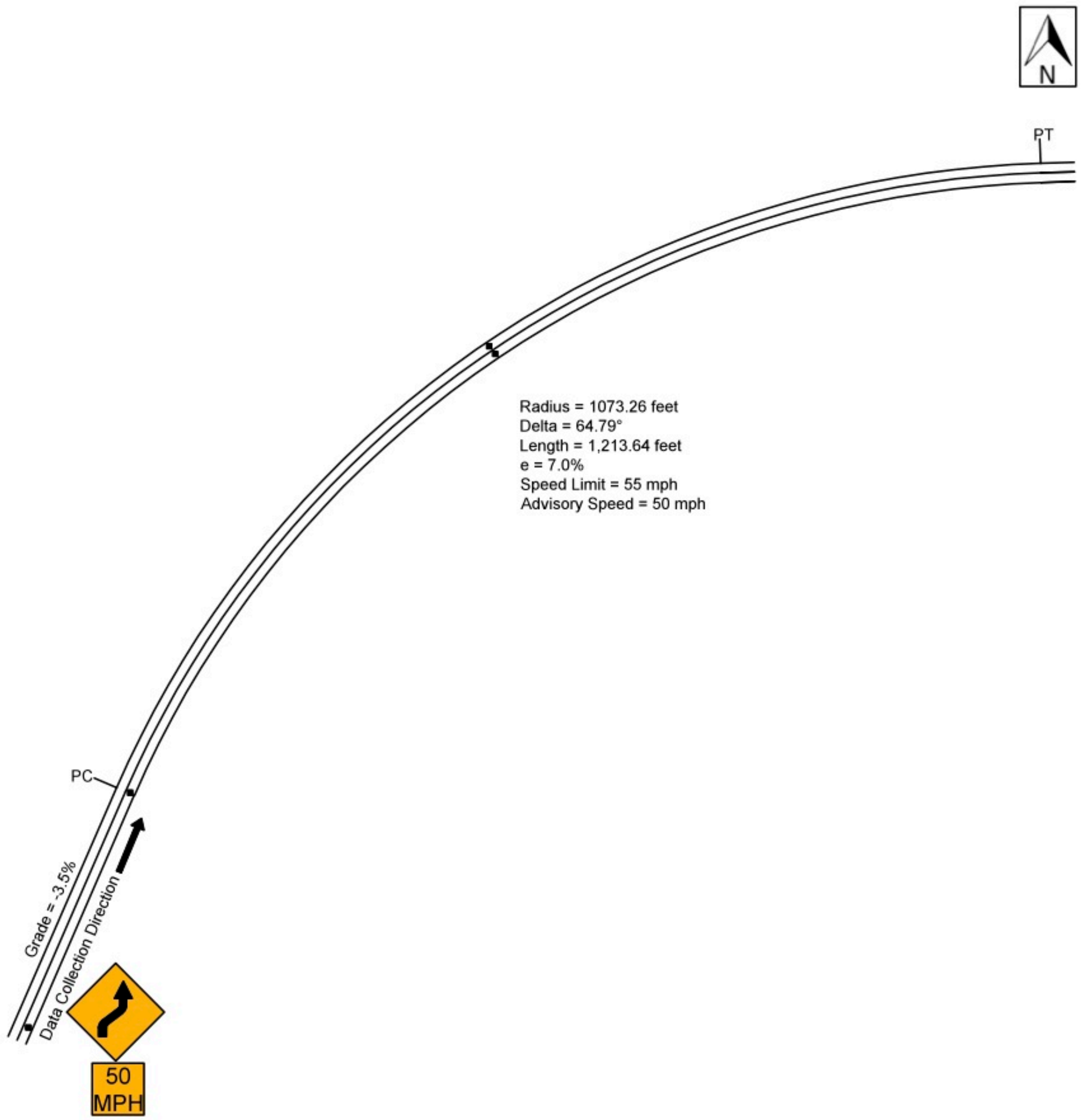


Figure 35. Diagram. Geometric layout of the WV Route 32 comparison site (not to scale).

Table 11 shows the speed data for the before period. A simple *t*-test was computed to compare the operating speeds at each data-collection location. For this comparison site, there were no statistically significant differences in speed measures.

Table 11. WV Route 32 comparison site before data.

Speed Metric	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N
Passenger cars	57.530	6.357	54.793	6.293	52.345	7.148	232
Heavy trucks	56.917	2.644	52.75	4.515	50.000	4.880	12
Daytime passenger cars	57.926	6.068	54.886	5.912	52.737	7.301	175
Nighttime passenger cars	56.917	7.114	54.446	7.444	51.107	6.627	56
Opposed passenger cars	58.581	5.870	54.774	5.812	52.452	7.348	31
Unopposed passenger cars	57.368	6.427	54.796	6.378	52.328	7.135	201
Free-flow passenger cars	57.623	6.381	54.912	6.387	52.265	7.402	204
Non free-flow passenger cars	56.857	6.246	53.929	5.591	52.929	4.981	28

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N = Number of Observations

Figure 36 graphically shows the PSL, advisory speed limit, and the mean speeds for passenger cars and trucks along the WV Route 32 comparison section during the before period. As shown, the truck and passenger car mean speeds are similar across the section, but the heavy truck mean speed is consistently about 2 mph less than the passenger car mean speed. The deceleration rates appear to be very similar. The dispersion of speed at the midpoint of the curve is much greater for passenger cars than at the other locations, which leads to the 85th percentile being relatively stable from the approach to the midpoint of the curve. The mean acceleration rate from the PC to the midpoint of the curve was -0.434 ft/s (1.38) for passenger cars and -0.615 ft/s (0.700) for heavy trucks.

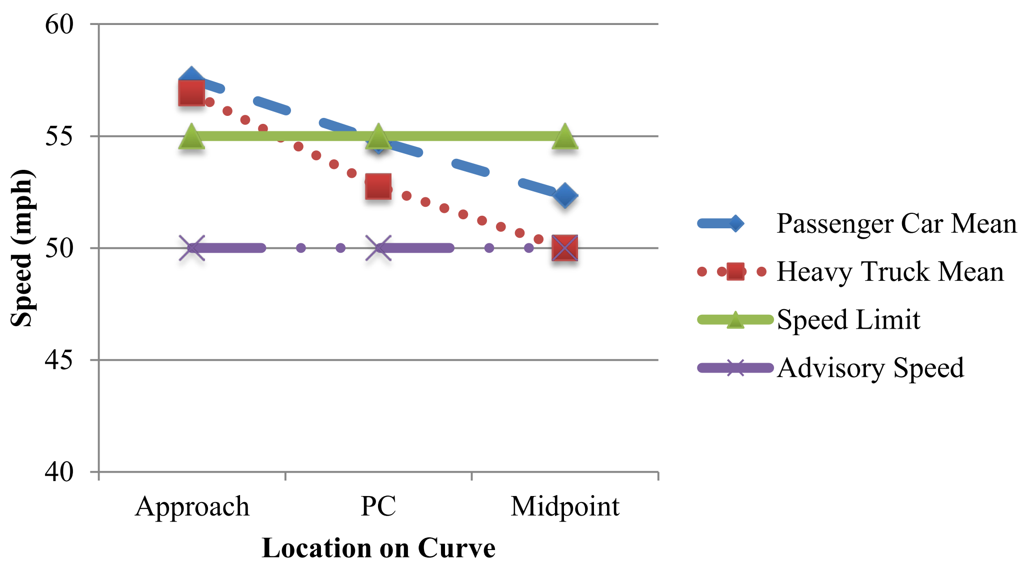


Figure 36. Graph. Graphical representation of speeds at the WV Route 32 comparison site.

The research team also collected friction data at eight locations within the comparison site. Table 12 presents the macro-texture, micro-texture, and SNs at the horizontal curve and approach locations. SNs within the curve were lower those on the approach tangent, which was expected because of the lateral friction demand by drivers traversing horizontal curves.

Table 12. WV Route 32 comparison site friction data.

Location	PC +300	PC + 200	PC + 100	PC	1/4	Mid	3/4	PT
CTM MPD	0.727	0.830	0.790	0.980	0.873	0.870	1.560	1.037
DFT 20	0.480	0.515	0.500	0.450	0.445	0.450	0.395	0.525
SN 65	0.380	0.410	0.400	0.360	0.360	0.360	0.280	0.420

CTM = Circular Texture Meter
MPD = Mean Profile Depth
DFT = Dynamic Friction Meter
SN = Skid Number
PC = Point of Curvature
PT = Point of Tangent

The research team collected encroachment data for 110 vehicles in the northbound direction and 129 vehicles in the southbound direction. Two vehicles encroached onto the shoulder, and no vehicles crossed the centerline in the northbound direction. Six vehicles crossed the centerline, and no vehicles encroached onto the shoulder in the southbound direction. Braking maneuvers were also observed for the northbound vehicles. Because the camera was located before the PC, vehicles braking before and within the curve were observed. At this site, 15 of the 110 vehicles were observed to brake before the curve and 10 of the 110 were observed to brake within the curve.

U.S. Route 33 Treatment Site (T2 in Table 6)

The second treatment site was located on U.S. Route 33 east of Harman, WV. The direction of travel for the data collection was in the eastbound direction. The radius of curve and the curve length, calculated using Google Earth™, were 209.97 ft and 539.22 ft, respectively. The superelevation for this curve was 6.0 percent, and the vertical grade was -8.0 percent. There was a substantial slope on the inside of the curve, which limited horizontal sight distance along the curve. The PSL was 55 mph. As figure 37 shows, there is a reverse turn warning sign (W1-3) with a 25 mph advisory speed plaque (W13-1P). The travel lanes were 11 ft wide, and there was a 4-ft shoulder on both sides.

Speed data were collected 300 ft before the PC, at the PC, and at the midpoint of the curve. The sensor was placed in the opposing direction of travel at the midpoint of the curve to determine whether vehicles were present in the opposing lane. In this case, there were two travel lanes in the westbound direction, and the sensor was placed in the left lane. The camera was placed upstream of the PC (approximately 200 ft) to allow for observation of braking before and into the curve for eastbound vehicles. Figure 37 shows the horizontal curve along with the speed data-collection locations.

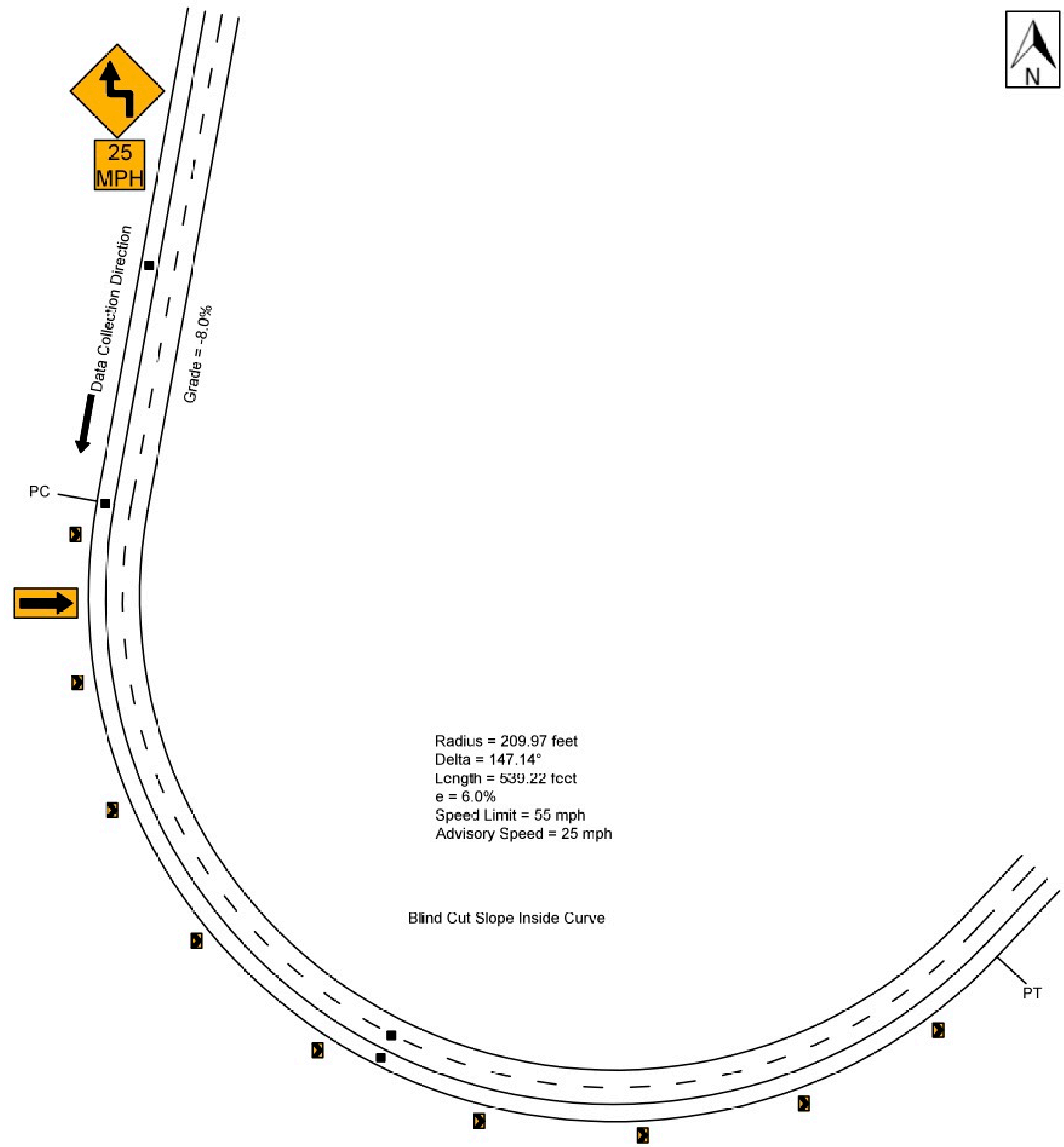


Figure 37. Diagram. Geometric layout of the U.S. Route 33 treatment site (not to scale).

Table 13 shows the speed data for the before period. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. Evaluation at this site showed statistically significant differences between passenger car and heavy truck speeds at all three locations. The difference was approximately 13 mph at the approach and 3.5 mph at the midpoint of the curve. Daytime speeds exceeded nighttime speeds for all locations, but the difference was only statistically significant at the PC. The relationship between free-flow speeds and non-free-flow speeds was inconsistent, and the difference was statistically significant at the PC.

Table 13. U.S. Route 33 treatment site before data.

Speed Metric	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N
Passenger cars	**47.413**	6.675	**39.120**	5.855	**31.982**	4.705	276
Heavy trucks	**34.265**	8.497	**32.118**	5.536	**28.441**	7.153	34
Daytime passenger cars	47.715	6.725	**39.729**	5.647	32.113	4.383	221
Nighttime passenger cars	46.200	6.384	**36.673**	6.086	31.455	5.843	55
Opposed passenger cars	47.970	7.760	38.788	6.637	31.982	4.705	33
Unopposed passenger cars	47.337	6.528	39.165	5.754	28.441	7.153	243
Free-flow passenger cars	47.332	6.747	**38.817**	5.656	32.127	4.709	229
Non-free-flow passenger cars	47.809	6.368	**40.596**	6.609	31.277	4.675	47

Bold indicates significance at the 95-percent confidence level (α=0.05).
SD = Standard Deviation
PC = Point of Curvature
N = Number of Observations

Figure 38 graphically shows the PSL, advisory speed limit, and the mean speeds for passenger cars and trucks along the U.S. Route 33 treatment section during the before period. As shown, the passenger car speeds decelerate substantially from the approach to the middle of the curve. The heavy truck mean speeds are more aligned with the 15th percentile passenger car speed, but there is less deceleration for heavy vehicles than passenger cars at this site. The truck speeds are consistent with the advisory speed of 25 mph at the midpoint of the curve. The mean acceleration rate from the PC to the midpoint of the curve was found to be -1.58 ft/s (0.817) for passenger cars and -1.075 ft/s (0.887) for heavy trucks.

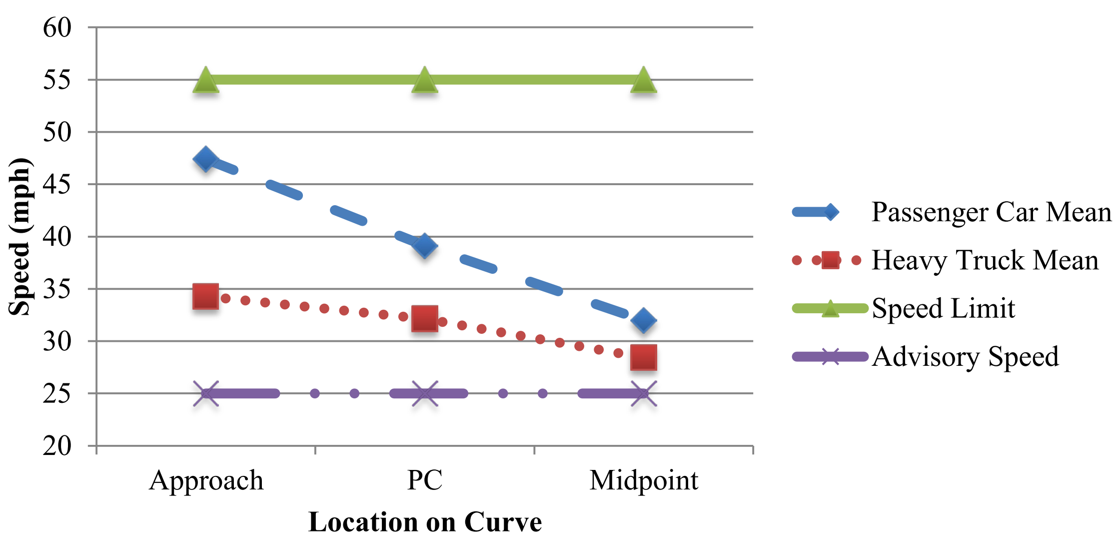


Figure 38. Graph. Graphical representation of speeds at the U.S. Route 33 treatment site.

Table 14 shows the macro-texture, micro-texture, and SNs for the U.S. Route 33 treatment site across the eight data-collection locations. The data show no discernible pattern because the friction and texture levels are fairly constant throughout the site.

Table 14. U.S. Route 33 treatment site friction data.

Location	PC +300	PC + 200	PC + 100	PC	1/4	Mid	3/4	PT
CTM MPD	0.703	1.137	0.937	0.877	0.910	0.717	1.060	0.703
DFT 20	0.415	0.435	0.425	0.505	0.405	0.390	0.405	0.460
SN 65	0.330	0.340	0.340	0.400	0.330	0.320	0.320	0.360

CTM = Circular Texture Meter
MPD = Mean Profile Depth
DFT = Dynamic Friction Meter
SN = Skid Number
PC = Point of Curvature
PT = Point of Tangent

Encroachment data were collected for 122 vehicles in the eastbound direction and 130 vehicles in the westbound direction. Six vehicles encroached onto the shoulder, and 15 vehicles crossed the centerline in the eastbound direction. Two vehicles crossed the centerline, and one vehicle encroached onto the shoulder in the westbound direction. Because there were two travel lanes in the westbound direction, it was observed that 48 vehicles crossed or straddled the broken line between the two lanes in the same direction. Braking maneuvers were also observed for the eastbound vehicles. Because the camera was located before the PC, vehicles braking before and within the curve were observed. At this site, 115 of the 122 vehicles were observed to brake before the curve, and 3 of the 122 vehicles were observed to brake within the curve.

U.S. Route 33 Comparison Site (C2 in Table 6)

Data for the U.S. Route 33 comparison were collected approximately 2 mi east of the treatment site. The direction of travel for the data collection was eastbound. The radius of curve and the curve length, calculated using Google Earth™, were 264.83 ft and 693.60 ft, respectively. The superelevation for this curve was 8.5 percent, and the vertical grade was -7.5 percent. There was a substantial cut-slope on the inside of the curve that limited horizontal sight distance along the curve. The PSL was 55 mph. As figure 39 shows, there was a reverse turn warning sign (W1-3) with a 25-mph advisory speed plaque (W13-1P). The travel lanes were 11 ft wide, and there was a 4-ft shoulder on both sides.

Speed data were collected 100 ft before the PC (because of the proximity of an adjacent horizontal curve), at the PC, and at the midpoint of the curve. The sensor in the opposing direction of travel was placed at the midpoint of the curve to whether vehicles were present in the opposing lane. The camera was placed upstream of the PC (approximately 200 ft) to allow observation of braking before and into the curve for eastbound vehicles. Figure 39 shows the horizontal curve along with the speed data-collection locations.

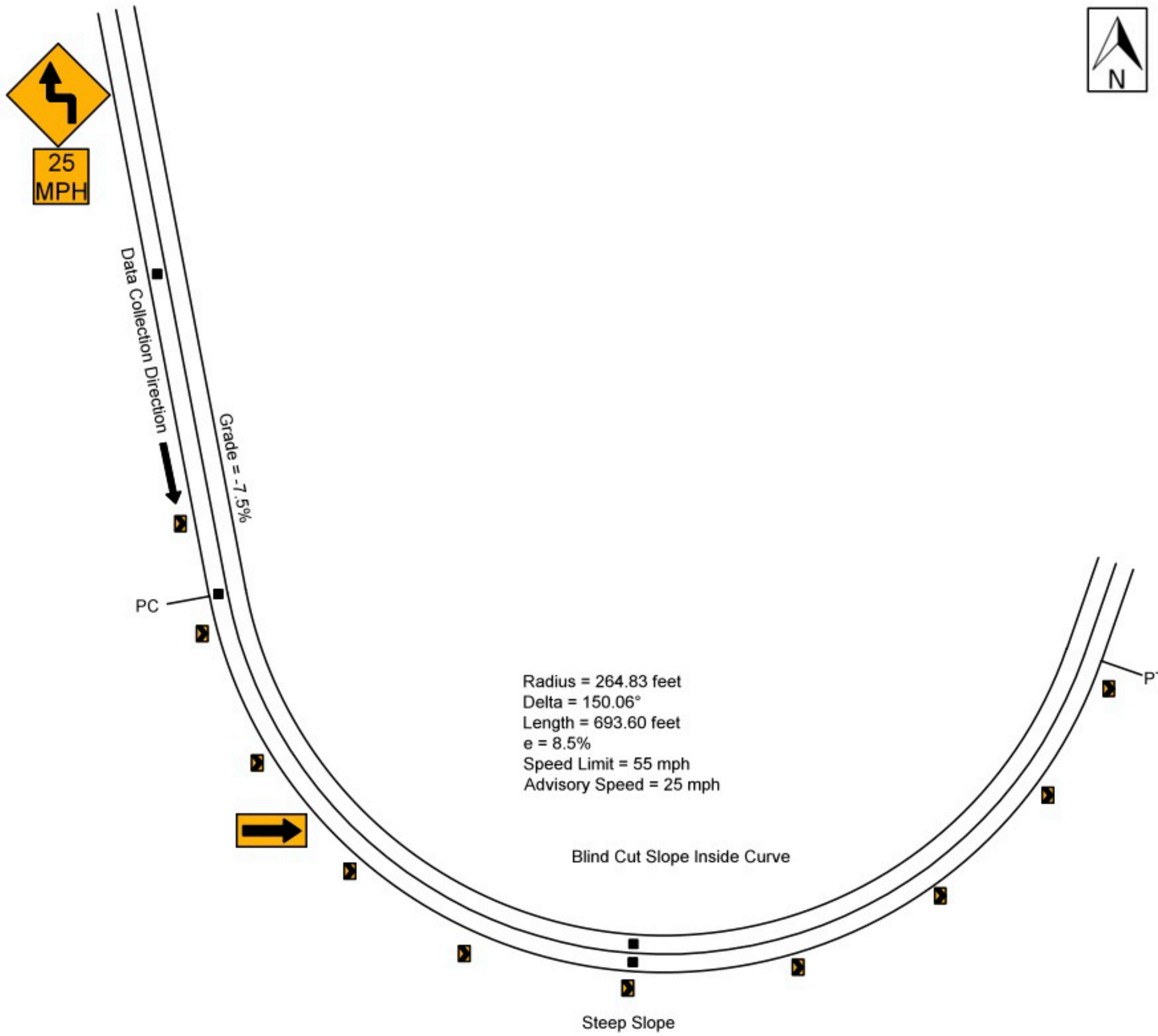


Figure 39. Diagram. Geometric layout of the U.S. Route 33 comparison site (not to scale).

Table 15 shows the speed data for the before period. The research team performed a simple *t*-test for each of the comparative measures at each of the curve locations. For this site, a statistically significant difference was found between passenger car and heavy truck speeds at all three locations. The difference was approximately 5.5 mph at the approach and PC locations, and 2.5 mph at the midpoint of the curve. The daytime speeds were greater than the nighttime speeds at all three locations, but the difference was only statistically significant at the PC. The free-flow speeds were greater than the non-free-flow speeds at all locations, but the difference was not statistically significant at the midpoint of the curve.

Table 15. U.S. Route 33 comparison site before data.

Speed Metric	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N
Passenger cars	**43.027**	5.966	**38.608**	6.566	**35.814**	6.528	263
Heavy trucks	**37.564**	8.012	**32.974**	7.354	**33.128**	5.908	39
Daytime passenger cars	43.376	5.804	**39.168**	6.362	36.056	6.633	197
Nighttime passenger cars	41.985	6.360	**36.939**	6.926	35.091	6.196	66
Opposed passenger cars	43.377	6.404	38.434	7.680	35.547	6.444	53
Unopposed passenger cars	42.938	5.864	38.652	6.274	35.881	6.562	210
Free-flow passenger cars	**43.404**	5.866	**39.207**	6.289	35.914	6.528	208
Non-free-flow passenger cars	**41.600**	6.181	**36.345**	7.142	35.436	6.786	55

Bold indicates significance at the 95-percent confidence level (α=0.05).
SD = Standard Deviation
PC = Point of Curve
N = Number of Observations

Figure 40 graphically shows the PSL, advisory speed limit, and the mean speeds for passenger cars and trucks along the U.S. Route 33 comparison section during the before period. As was the case at the treatment site, the mean speed for passenger cars decreased across the study section at a greater rate than the heavy trucks. However, the mean heavy truck speed stabilized at about 33 mph, which is 8 mph greater than the advisory speed. The 85th percentile speed for passenger cars was approximately the PSL (55 mph) at the approach and decreased to about 49 mph at the midpoint of the curve. The 15th percentile speed was approximately the advisory speed of 25 mph at the midpoint of the curve. There does not appear to be a relationship between passenger car speeds and heavy truck speeds at this site. The mean acceleration rate from the PC to the midpoint of the curve was found to be -0.679 ft/s (1.664) for passenger cars and -0.262 ft/s (1.468) for heavy trucks.

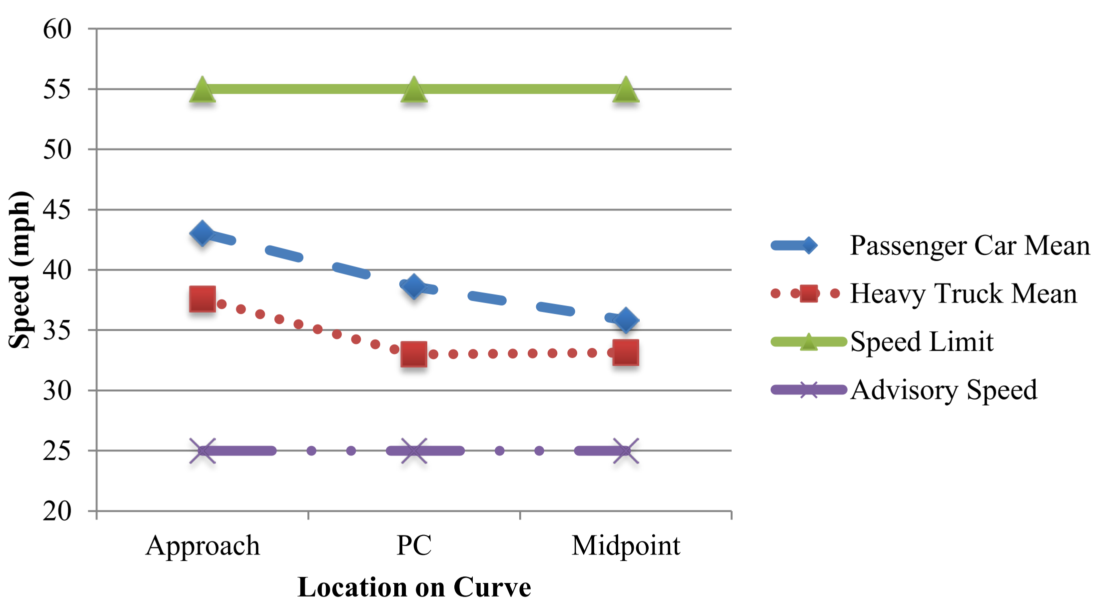


Figure 40. Graph. Graphical representation of speeds at the U.S. Route 33 comparison site.

Table 16 shows the macro-texture, micro-texture, and SNs for the U.S. Route 33 comparison site across the seven data-collection locations. It is clear from the data that both the macro-texture and micro-texture are very low at this site. This leads to a range of SNs from a low of 0.26 at the three-quarter point of the curve to a high of 0.34 at the PC. Similar to the treatment site, the comparison site is located on a very steep grade (approximately 8 percent). The steep grade creates the need for braking throughout the entire treatment and comparison sites, which is likely the reason that the pavement texture is consistent, but low, throughout the study site.

Table 16. U.S. Route 33 comparison site friction data.

Location	PC + 200	PC + 100	PC	1/4	Mid	3/4	PT
CTM MPD	0.463	0.530	0.547	0.420	0.350	0.327	0.430
DFT 20	0.355	0.400	0.450	0.360	0.400	0.365	0.380
SN 65	0.28	0.31	0.34	0.28	0.28	0.26	0.29

CTM = Circular Texture Meter
MPD = Mean Profile Depth
DFT = Dynamic Friction Meter
SN = Skid Number
PC = Point of Curvature
PT = Point of Tangent

The research team collected encroachment data for 89 vehicles in the eastbound direction and 92 vehicles in the westbound direction. Four vehicles encroached onto the shoulder, and 10 vehicles crossed the centerline in the eastbound direction. Five vehicles crossed the centerline, and 12 vehicles encroached onto the shoulder in the westbound direction. Braking maneuvers were also observed for the eastbound vehicles. Because the camera was located before the PC, vehicles braking before and within the curve were observed. At this site, 85 of the 89 vehicles were observed to brake before the curve, and 2 of the 89 vehicles were observed to brake within the curve.

U.S. Route 219 Treatment Site A—Mile Marker 6.32 (T3 in Table 6)

Data at treatment site A on U.S. Route 219 were collected in the northbound direction of travel. The radius of curve and the curve length, calculated using Google Earth™, were 605.08 ft and 753.82 ft, respectively. The superelevation for this curve was 11 percent, and the vertical grade was -2.0 percent. There was a roadside slope on the inside of the curve that limited horizontal sight distance along the curve. The PSL was 55 mph. As figure 41 shows, there was a warning sign (W1-2) of a curve to the right with a 30 mph advisory speed plaque (W13-1P). The travel lanes were 10 ft wide, and there was a 1-ft paved shoulder on both sides.

Speed data were collected 300 ft before the PC, at the PC, and at the midpoint of the curve. The sensor in the opposing direction of travel was placed at the midpoint of the curve to determine whether vehicles were present in the opposing lane. The camera was placed upstream of the PC (approximately 200 ft) to allow observation of braking before and into the curve for northbound vehicles. Figure 41 shows a graphical layout of the horizontal curve along with the speed data-collection locations.

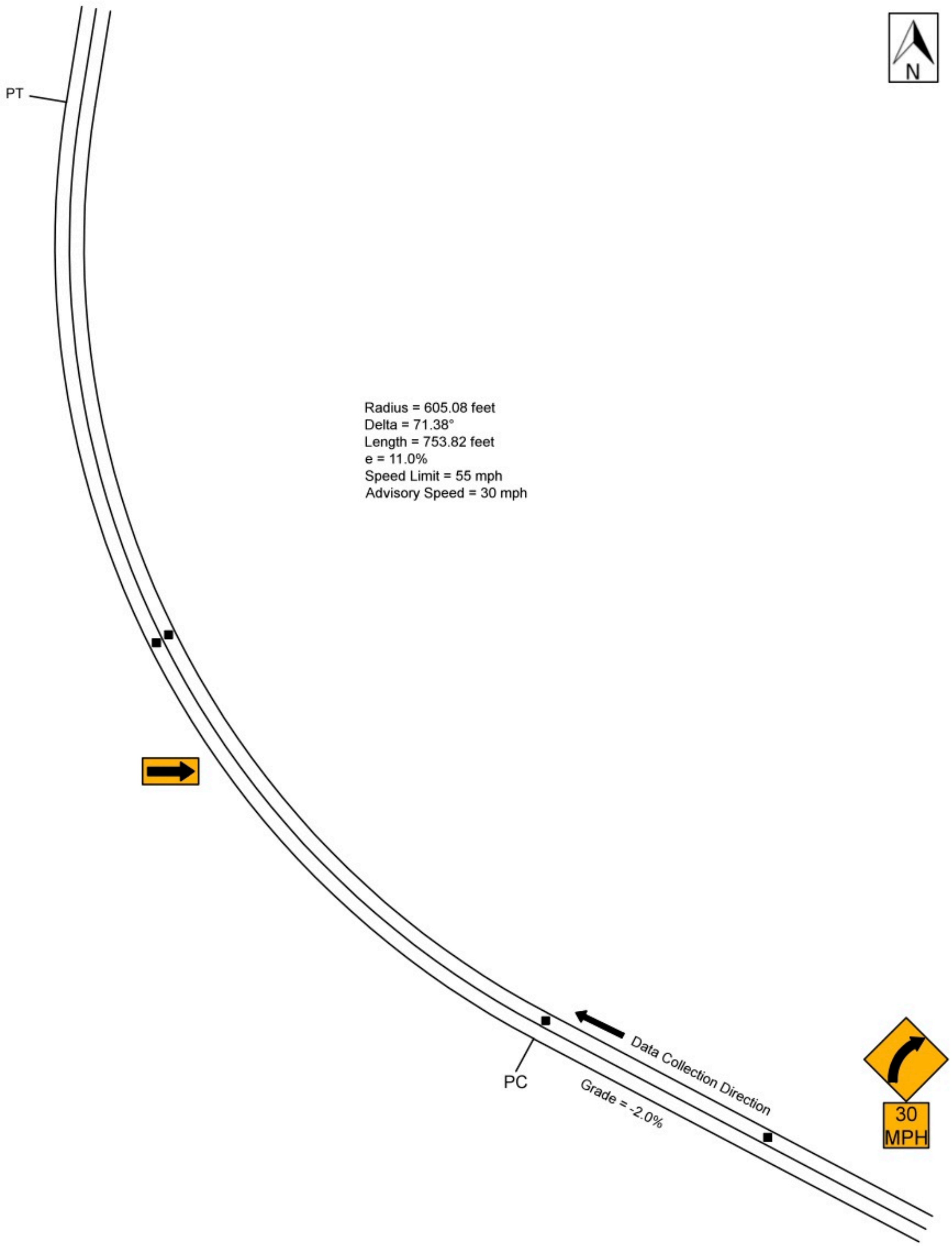


Figure 41. Diagram. Geometric layout of the U.S. Route 219 treatment A site (not to scale).

Table 17 presents the speed data for the before period. The research team completed a simple *t*-test for each of the speed measures at each of the curve locations. For this site, passenger car speeds were greater than the heavy truck speeds at all three locations. The difference was statistically significant at the approach location only. Daytime and nighttime speeds were statistically significant at the approach; however, there were only 14 vehicles observed in the nighttime period. At the approach and PC, the nighttime speeds were greater than the daytime speeds. The daytime speeds were greater (although insignificantly) that nighttime speeds at the midpoint of the curve.

Table 17. U.S. Route 219 treatment A site before data.

Speed Metric	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N
Passenger cars	**48.125**	6.058	46.397	6.062	36.294	7.532	136
Heavy trucks	**43.944**	5.826	45.222	5.287	34.389	4.629	18
Daytime passenger cars	**47.893**	6.172	46.336	6.191	36.434	7.356	122
Nighttime passenger cars	**50.143**	4.655	46.929	4.953	35.071	9.144	14
Opposed passenger cars	46.889	6.615	46.167	6.947	37.667	9.172	18
Unopposed passenger cars	48.314	5.977	46.432	5.948	36.085	7.273	118
Free-flow passenger cars	48.383	6.059	46.600	6.039	36.565	7.542	115
Non-free-flow passenger cars	46.714	6.282	45.286	6.214	34.810	7.481	21

Bold indicates significance at the 95-percent confidence level (α=0.05).
SD = Standard Deviation
PC = Point of Curvature
N = Number of Observations

Figure 42 graphically shows the PSL, advisory speed limit, and the mean speeds for passenger cars and trucks along U.S. Route 219 treatment site A during the before period. The figure shows that mean speeds for both heavy trucks and passenger cars remain relatively stable from the approach to the PC, but decrease substantially from the PC to the midpoint of the curve. At this site, it is clear that the mean speeds for both passenger cars and heavy trucks are similar across the study segment. The mean speeds are approximately 5 mph greater than the advisory speed of 30 mph at the midpoint of the curve. It is also clear that the dispersion of speed decreases substantially within the horizontal curve. The mean acceleration rate from the PC to the midpoint of the horizontal curve was -3.178 ft/s (2.959) for passenger cars and -3.042 ft/s (2.811) for heavy trucks.

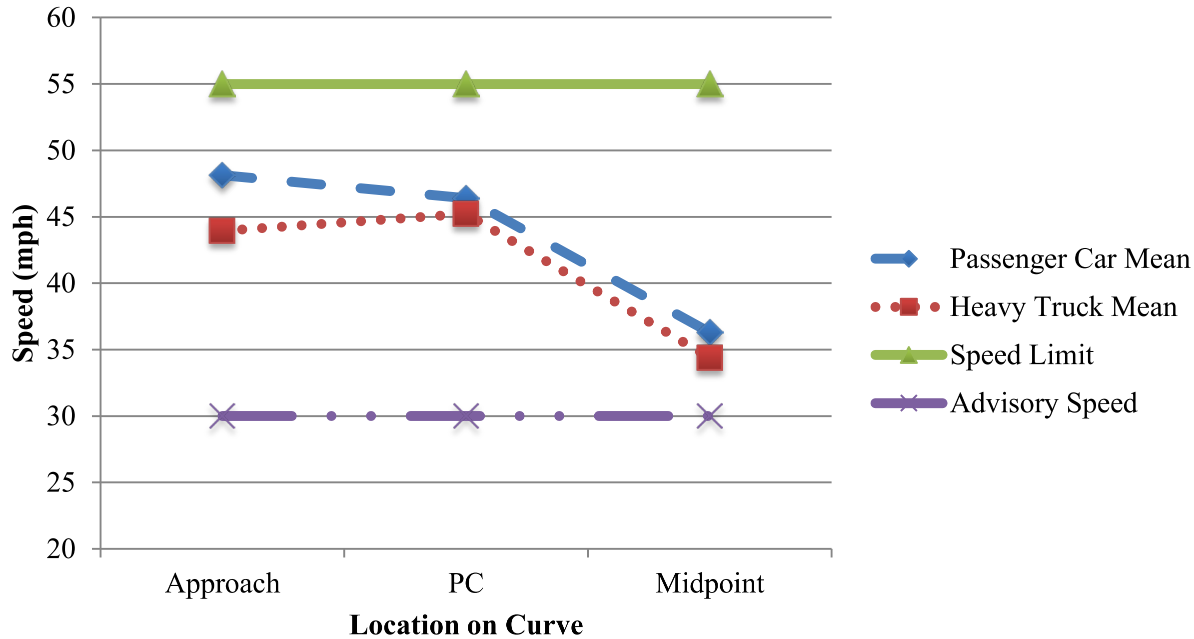


Figure 42. Graph. Graphical representation of speeds at the U.S. Route 219 treatment A site.

Table 18 shows the macro-texture, micro-texture, and SNs for the U.S. 219 treatment A site across the eight data-collection locations. From the table, it is clear that the mean profile depth is much greater across the entire site than the other data-collection sites. Greater mean-profile depth is associated with a lower SN. Conversely, the micro-texture friction is higher than most sites. The SN appears to be fairly constant from the approach throughout the horizontal curve.

Table 18. U.S. Route 219 treatment A site friction data.

Location	PC +300	PC + 200	PC + 100	PC	1/4	Mid	3/4	PT
CTM MPD	1.407	1.417	1.407	1.277	0.890	0.950	1.093	1.140
DFT 20	0.500	0.490	0.525	0.530	0.505	0.465	0.585	0.545
SN 65	0.380	0.370	0.360	0.400	0.360	0.380	0.370	0.400

CTM = Circular Texture Meter
MPD = Mean Profile Depth
DFT = Dynamic Friction Meter
SN = Skid Number
PC = Point of Curvature
PT = Point of Tangent

Encroachment data were collected for 77 vehicles in the northbound direction and 92 vehicles in the southbound direction. Two vehicles encroached onto the shoulder, and no vehicles crossed the centerline in the northbound direction. Six vehicles crossed the centerline, and three vehicles encroached onto the shoulder in the southbound direction. Braking maneuvers were also observed for the northbound vehicles. Because the camera was located before the PC, vehicles braking before and within the curve were observed. At this site, 56 of the 77 vehicles were observed braking before the curve, and 12 of the 77 vehicles were observed braking within the curve.

U.S. Route 219 Treatment Site B—Mile Marker 5.81 (T4 in Table 6)

The research team also collected at a second treatment site along U.S. Route 219. The direction of travel for the data collection was northbound. The radius of curve and the curve length, calculated using Google Earth™, were 272.85 ft and 673.42 ft, respectively. The superelevation for this curve was 12 percent, and the vertical grade was -3.0 percent. There was a roadside slope on the inside of the curve that limited horizontal sight distance along the curve. The travel lanes were 10.5 ft wide, and there was a 1-ft paved shoulder on both sides. The PSL is 55 mph. As Figure 44 shows, there was a warning sign (W1-2) of a curve to the right with a 25-mph advisory speed plaque (W13-1P).

Speed data were collected 300 ft before the PC, at the PC, and at the midpoint of the curve. The sensor in the opposing direction of travel was placed at the midpoint of the curve to determine whether vehicles were present in the opposing lane. The camera was placed upstream of the PC (approximately 200 ft) to observe braking before and into the curve for northbound vehicles. Figure 43 shows the layout of the horizontal curve along with the speed data-collection locations.

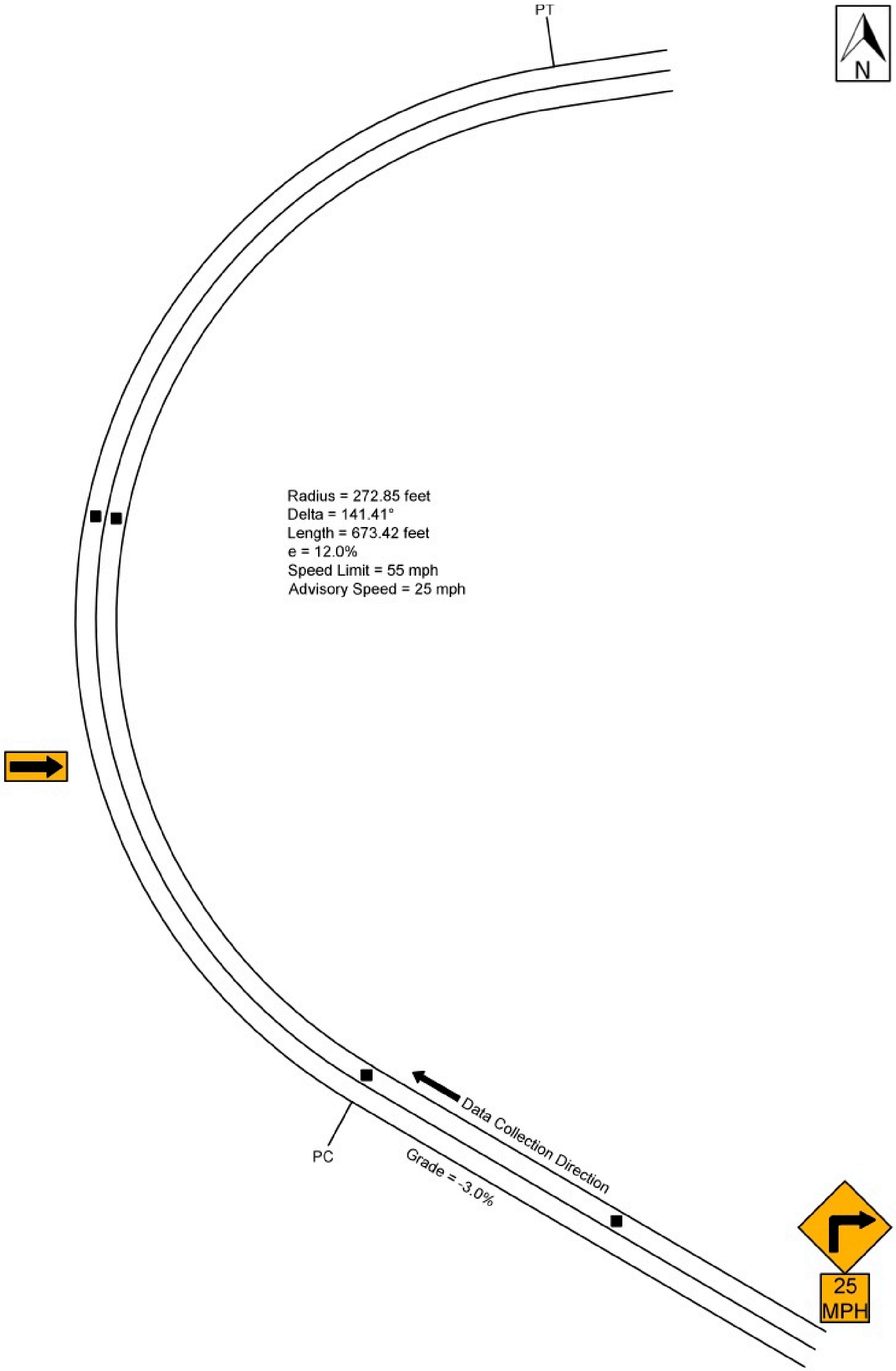


Figure 43. Diagram. Geometric layout of the U.S. Route 219 treatment B site (not to scale).

Table 19 presents speed data for the before period. A simple *t*-test was computed for each of these comparisons at each of the curve locations. For this site, passenger car speeds were greater than the heavy truck speeds at all three locations. The difference was statistically significant at the PC only. The unopposed passenger car speeds were greater than the opposed passenger car speeds at all three locations. Free-flow speeds were statistically different than non-free-flow speeds at the PC only.

Table 19. U.S. Route 219 treatment B site before data.

Speed Metric	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N
Passenger cars	48.661	8.304	**44.724**	6.720	34.622	6.285	127
Heavy trucks	46.684	7.616	**39.263**	6.145	33.474	4.823	19
Daytime passenger cars	48.848	8.082	44.857	6.881	34.464	6.373	112
Nighttime passenger cars	47.267	10.018	43.733	5.457	35.800	5.647	15
Opposed passenger cars	46.769	5.085	44.615	4.445	33.154	4.616	13
Unopposed passenger cars	48.877	8.584	44.737	6.946	34.789	6.443	127
Free-flow passenger cars	48.413	8.647	**45.298**	6.914	34.712	6.406	104
Non-free-flow passenger cars	49.783	6.578	**42.130**	5.120	34.217	5.823	23

Bold indicates significance at the 95-percent confidence level (α=0.05).
SD = Standard Deviation
PC = Point of Curve
N = Number of Observations

Figure 44 graphically shows the PSL, advisory speed limit, and the mean speeds for passenger cars and trucks along U.S. Route 219 treatment site B during the before period. The figure shows that passenger cars decelerate at a much greater rate within the horizontal curve compared with the approach to the horizontal curve. Heavy trucks appear to have a more constant deceleration from the approach to the midpoint of the horizontal curve, which explains the significant difference at the PC location. Mean speeds are about 7 mph less than the PSL at the approach and about 10 mph greater than the advisory speed at the midpoint of the horizontal curve. The 15th percentile speed is approximately equal to the advisory speed at the midpoint of the horizontal curve. The mean acceleration rate from the PC to the midpoint of the curve was -2.603 ft/s (2.064) for passenger cars and -1.645 ft/s (1.53) for heavy vehicles.

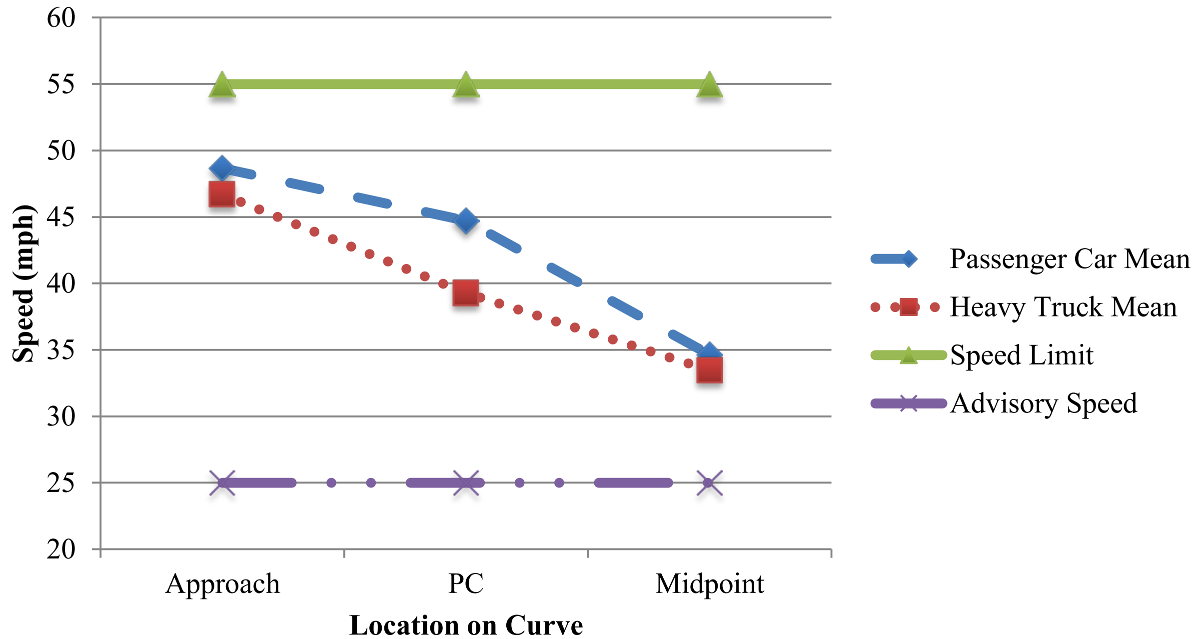


Figure 44. Graph. Graphical representation of speeds at the U.S. Route 219 treatment B site.

Table 20 shows the macro-texture, micro-texture, and SNs for the U.S. Route 219 treatment B site across the eight data-collection locations. Again, there is no discernible pattern of the SN from the approach to the end of the site, but the SN is pretty high compared with many of the other sites. The mean-profile depth is small (ranging from 0.493 to 0.823), and the micro-texture friction is high (ranging from 0.465 to 0.585).

Table 20. U.S. Route 219 treatment B site friction data.

Location	PC +300	PC + 200	PC + 100	PC	1/4	Mid	3/4	PT
CTM MPD	0.493	0.493	0.647	0.680	0.823	0.570	0.643	0.573
DFT 20	0.535	0.560	0.525	0.530	0.505	0.465	0.585	0.545
SN 65	0.380	0.400	0.400	0.410	0.400	0.350	0.440	0.400

CTM = Circular Texture Meter
MPD = Mean Profile Depth
DFT = Dynamic Friction Meter
SN = Skid Number
PC = Point of Curvature
PT = Point of Tangent

The research team collected encroachment data for 61 vehicles in the northbound direction and 61 vehicles in the southbound direction. Ten vehicles encroached onto the shoulder, and 10 vehicles crossed the centerline in the northbound direction. Twelve vehicles crossed the centerline, and 2 vehicles encroached onto the shoulder in the southbound direction. Braking maneuvers were also observed for the northbound vehicles. Because the camera was located before the PC, vehicles braking before and within the curve were observed. At this site, 31 of the 61 vehicles were observed braking before the curve, and 20 of the 61 vehicles were observed braking within the curve.

U.S. Route 219 Comparison Site (C3/C4 in Table 6)

The U.S. Route 219 comparison site was located approximately 5 mi north of the two treatment sites on the same route. The direction of travel for the data collection was northbound. The curve radius and the curve length, calculated using Google Earth™, were 544.57 ft and 826.91 ft, respectively. The superelevation for this curve was 12 percent, and the vertical grade was -6.5 percent. The PSL was 55 mph. As figure 45 shows, there was a reverse curve warning sign (W1-4) with a 30-mph advisory speed plaque (W13-1P). The travel lanes were 11 ft wide, and there was no paved shoulder.

Speed data were collected 100 ft before the PC (because of an adjacent horizontal curve), at the PC, and at the midpoint of the curve. The research team placed the sensor in the opposing direction of travel at the midpoint of the curve to determine whether vehicles were present in the opposing lane. The camera was placed at the PC to observe braking into the curve for northbound vehicles. Figure 45 shows a layout of the horizontal curve along with the speed data-collection locations.

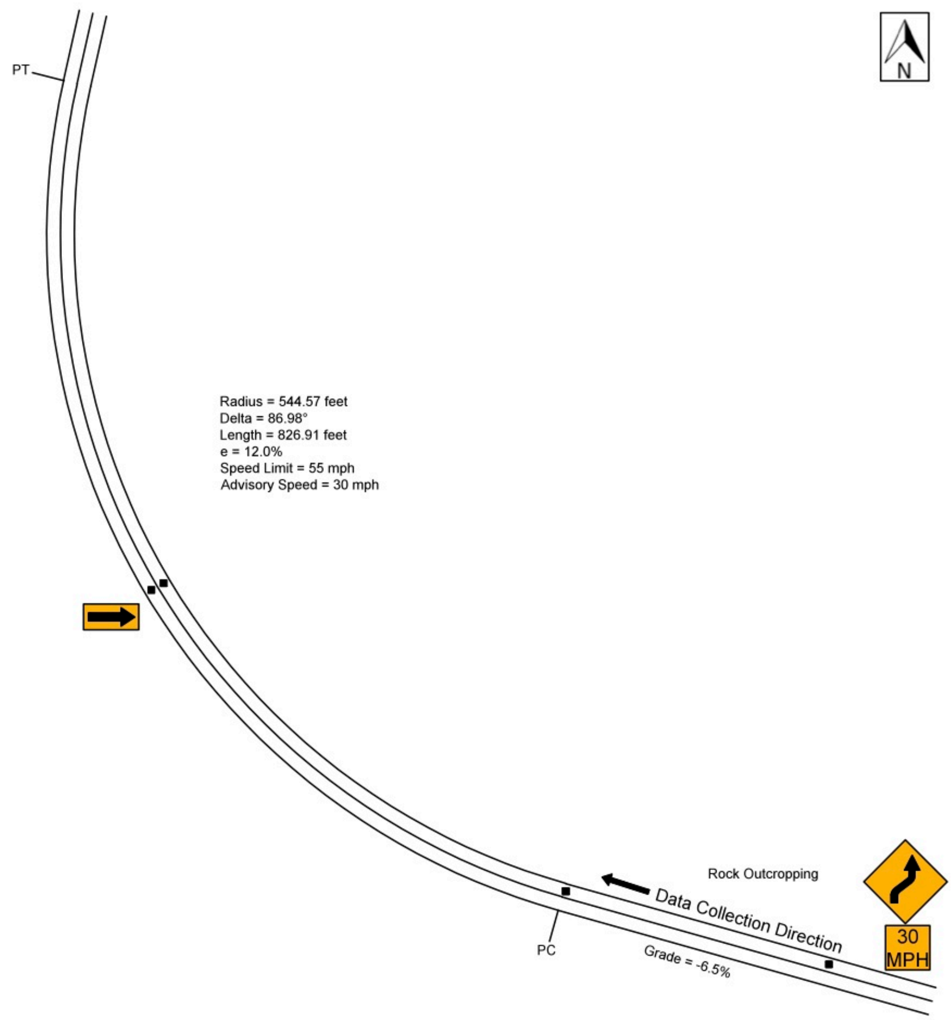


Figure 45. Diagram. Geometric layout of the U.S. Route 219 comparison site (not to scale).

Table 21 presents the speed data for the before period. The research team computed a simple *t*-test for each of the speed measures at each of the curve locations. The passenger car speeds were greater than the heavy truck speeds at all three locations. This difference was statistically significant at all three locations, even with only 79 and 13 passenger car and truck speed observations, respectively. The daytime speeds were greater than the nighttime speeds at all three sites. This difference was not statistically significant at any site, likely owing to the small sample size. The opposed passenger car speeds were greater than the unopposed passenger car speeds at every site, and were statistically significant at the approach. The free-flow speeds were greater than the non-free-flow speeds at every site, but not statistically significant at any location.

Table 21. U.S. Route 219 comparison site before data.

Speed Metric	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N
Passenger cars	**44.506**	6.961	**42.025**	6.653	**41.152**	5.371	79
Heavy trucks	**39.154**	7.186	**38.154**	5.886	**36.000**	6.506	13
Daytime passenger cars	44.647	6.083	42.176	5.122	41.426	4.566	68
Nighttime passenger cars	43.634	11.360	41.091	12.973	39.455	9.038	11
Opposed passenger cars	**46.000**	4.219	43.273	5.442	42.909	5.594	11
Unopposed passenger cars	**43.444**	7.482	41.235	6.799	40.086	5.771	81
Free-flow passenger cars	44.714	7.051	42.100	6.740	41.157	5.468	70
Non free-flow passenger cars	42.889	6.353	41.444	6.267	41.111	4.833	9

Bold indicates significance at the 95-percent confidence level (α=0.05).
SD = Standard Deviation
PC = Point of Curvature
N = Number of Observations

Figure 46 graphically shows the PSL, advisory speed limit, and the mean speeds for passenger cars and trucks along the U.S. Route 219 comparison site during the before period. The passenger car and heavy truck speeds remain fairly constant from the approach to the midpoint of the horizontal curve. This is likely because this horizontal curve is in the middle of several horizontal curves with no long tangent sections between. The mean passenger car speed was approximately 12 mph greater than the advisory speed, and the heavy truck speed was approximately 10 mph greater. The 85th percentile operating speed was approximately equal to the PSL (55 mph). The mean acceleration rate from the PC to the midpoint of the curve was -0.213 ft/s (1.201) for passenger cars and -0.157 ft/s (1.627) for heavy trucks.

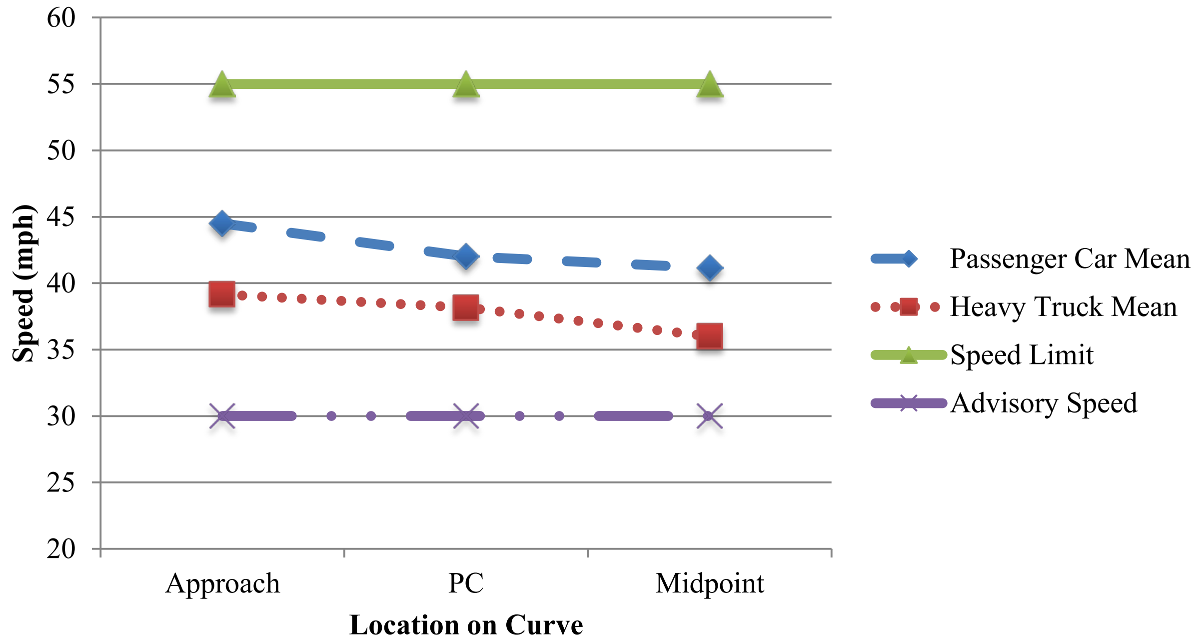


Figure 46. Graph. Graphical representation of speeds at the U.S. Route 219 comparison site.

Table 22 shows the macro-texture, micro-texture, and SNs for the U.S. Route 219 comparison site across the six data-collection locations. As the table shows, the SN is quite low across the site and is somewhat uniform from the approach to the end of the horizontal curve. Again, the site is on a fairly substantial grade (about 6.5 percent downhill), which may contribute to increased longitudinal friction as a result of braking.

Table 22. U.S. Route 219 comparison site friction data.

Location	PC + 100	PC	1/4	Mid	3/4	PT
CTM MPD	0.650	0.760	0.457	0.520	0.610	0.683
DFT 20	0.390	0.395	0.385	0.390	0.410	0.390
SN 65	0.310	0.320	0.290	0.300	0.330	0.320

CTM = Circular Texture Meter
MPD = Mean Profile Depth
DFT = Dynamic Friction Meter
SN = Skid Number
PC = Point of Curvature
PT = Point of Tangent

Encroachment data were collected for 49 vehicles in the northbound direction and 60 vehicles in the southbound direction. Eight vehicles encroached onto the shoulder, and no vehicles crossed the centerline in the northbound direction. Twenty vehicles crossed the centerline, and 3 vehicles encroached onto the shoulder in the southbound direction. Braking maneuvers were also observed for the northbound vehicles. Because the camera was located at the PC, vehicles braking maneuvers within the curve were observed. At this site, 47 of the 49 vehicles were observed to brake within the curve.

COMPARISON OF BEFORE AND AFTER PERIOD DATA

The HFST was applied to the four treatment site locations in June 2012, and the research team observed speed, encroachment, and braking maneuvers in August 2012. The second after period data-collection effort was completed in June 2013, and included speed, encroachment, and braking maneuver field measurements and observation. This section of the report describes the differences found between the before and after conditions at the treatment sites, as well as the comparison sites. If significant differences were found from the before to after condition at the comparison sites, then those differences were considered in the analysis of the treatment sites.

As described in the data collection and analysis methods section above, the research team excluded all vehicles that were not passenger cars and that were not considered free-flow vehicles from the analysis database. The data analysis files also included indicators for when vehicles were present opposite to the travel lane where the operational evaluation was being conducted. The mean speed, change in mean speed between the PC and midpoint of the curve, percentage of vehicles exceeding the PSL, encroachment, and friction analysis results are all included in this section of the report.

Mean Operating Speed Results

West Virginia Route 32 (Treatment and Comparison Site)

Table 23 shows the before and after mean speeds and speed deviations for the treatment and comparison sites on WV Route 32. The table shows the values for both passenger cars and heavy trucks at the approach, PC, and curve midpoint. The mean and SDs are given for free-flow vehicles with no opposing vehicles in the opposite direction because very few vehicles were present in the opposing lane during the analysis period (see before data period data summary above) and because these mean speeds were seldom statistically different in the before period data analysis. Only daytime speeds were reported for comparison in the before and after periods because too few nighttime observations were available or because passenger car and truck speeds seldom varied in the before period analysis. As table 23 shows, there was no statistically significant change in speed measures for the comparison site from the before to the first after period. However, significant reductions in speed were found at the PC and curve midpoints for passenger cars at the treatment site. In both cases, the reduction was approximately 1.5 mph. The midpoint speed was found to be significantly lower for heavy trucks, by approximately 8 mph, in the first after period when compared with the before period, but this was likely the result of the very small number of trucks included in the sample. When comparing the second after period to the first after period, the mean truck speeds increased to levels similar to the before period.

There was a significant reduction in speed for passenger cars at the approach point in the second after period, when compared with the before period, at the treatment site. However, there was also a significant reduction at the approach point for the comparison site. When taking this reduction into account, the reduction in speed for the treatment site was no longer statistically significant. The mean speed of passenger cars at the PC and curve midpoint increased slightly in the second after period, but they were still less than the before period. There was a 4.2-mph reduction in truck speed at the approach of the treatment site, but this reduction did not continue through the curve. There was also a statistically significant increase in truck speeds at the curve midpoint in the second after period compared with the first. The increase was approximately

6.9 mph, but the speed was still 1.3 mph lower than the before period. The truck speed at the midpoint during the first after period was low when compared with the before and second after period, but there were 12 or fewer trucks in each observation period.

Table 23. Operating speeds at WV Route 32 sites before and after treatment.

Speed Metric	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N
Treatment PCs before	55.216	7.3	53.743	6.283	48.054	5.639	167
Treatment PCs 1st after	54.166	7.959	**52.337**	7.343	**46.554**	5.126	175
Treatment PCs 2nd after	<u>**53.111**</u>	8.447	52.849	6.645	47.633	7.212	199
Treatment trucks before	56.50	3.987	51.50	5.089	49.833	4.622	6
Treatment trucks after	55.125	3.227	50.50	5.855	**41.625**	4.658	8
Treatment trucks 2nd after	<u>**52.333**</u>	6.443	51.417	5.384	***48.50***	3.477	12
Comparison PCs before	57.839	6.04	55.04	6.046	52.758	7.617	124
Comparison PCs 1st after	57.443	5.317	54.584	5.057	52.227	5.587	185
Comparison PCs 2nd after	***56.504***	6.315	55.417	6.923	52.483	6.06	240
Comparison trucks before	56.429	2.992	53.857	2.734	49.429	3.823	7
Comparison trucks 1st after	57.889	8.023	52.556	5.94	50.667	5.148	9
Comparison trucks 2nd after	54.375	6.811	51.938	6.708	49.25	3.474	16

Bold indicates significance at the 95-percent confidence level (α=0.05) when comparing either after period to the before period.
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods.
Underline indicates that, when accounting for the speed change at the comparison site, the change in the speed at the treatment site was not statistically significant at the 95-percent confidence level (α = 0.05).
SD = Standard Deviation
PC = Point of Curvature
N = Number of Observations

Figure 47 and figure 48 show graphical representations of the speed profiles for the WV Route 32 treatment and comparison sites, respectively. The general shape of the speed profiles for the three collection periods remained relatively similar.

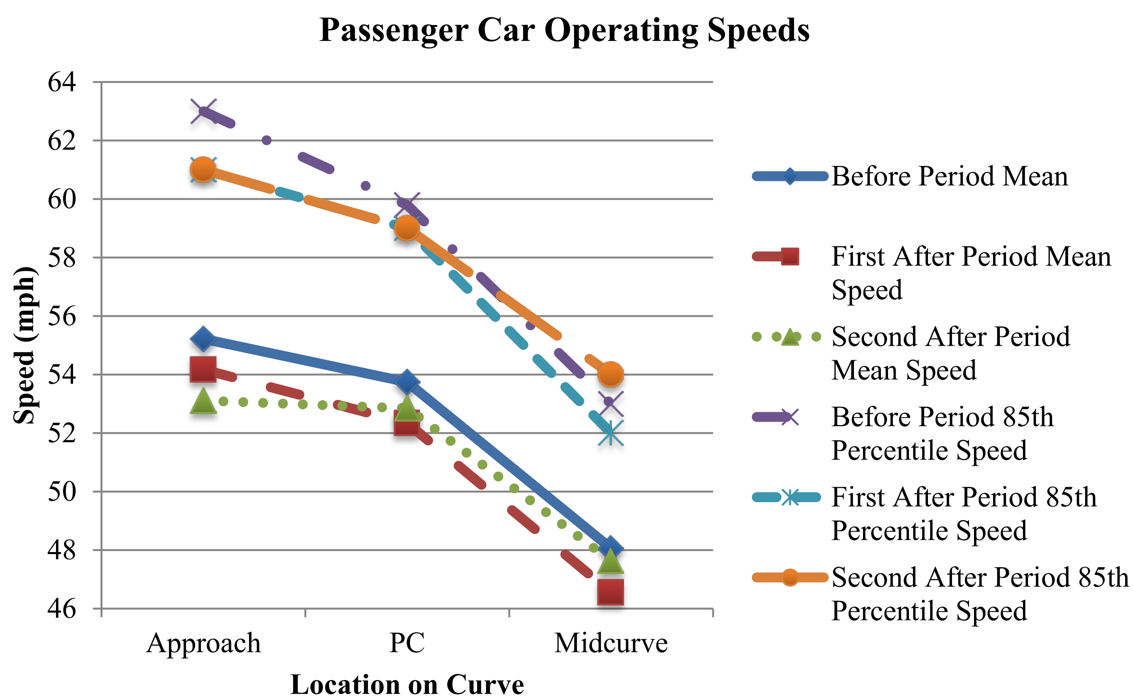


Figure 47. Graph. WV Route 32 treatment site operating speeds (PSL = 55 mph).

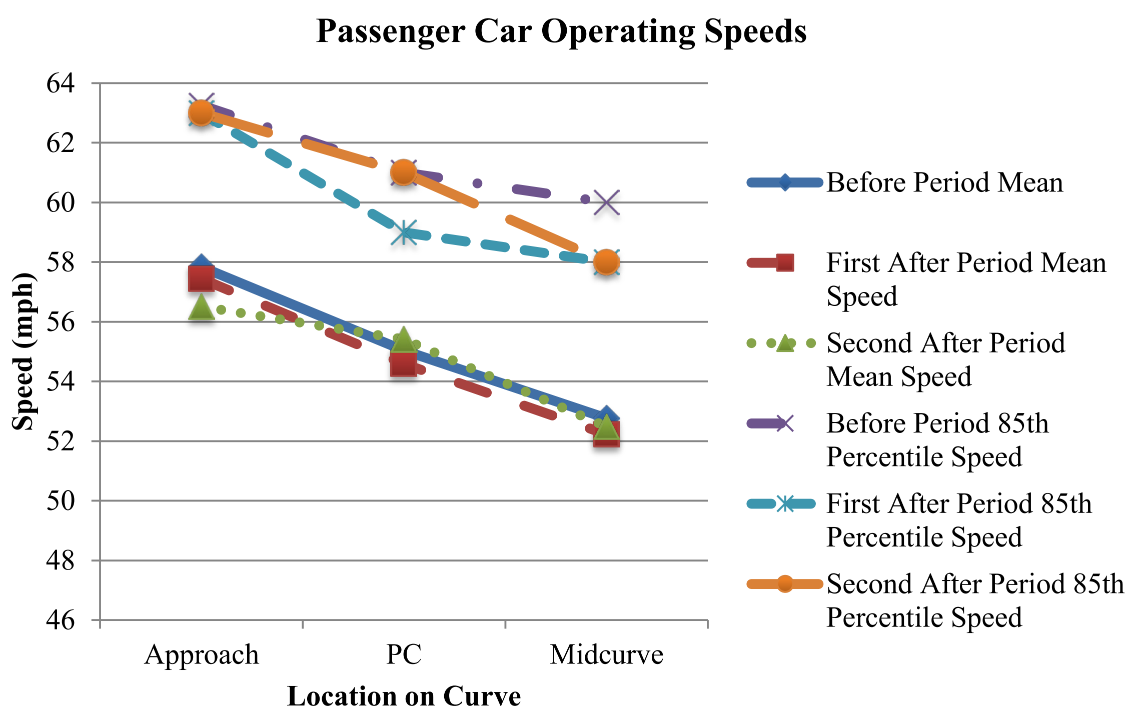


Figure 48. Graph. WV Route 32 comparison site operating speeds (PSL = 55 mph).

U.S. Route 33 (Treatment and Comparison Site)

Table 24 shows the before and after mean operating speeds for the treatment and comparison sites on U.S. Route 33. Again, there was no statistically significant change in mean speeds for both passenger cars and heavy trucks at the comparison from the before period to the first after period. The difference in mean speed for passenger cars between the before treatment application and first after treatment application were statistically significant at all three data-collection points. This difference shows reduced speed from the before to the after period at the treatment location. The change was approximately 1.3 mph at the approach and PC locations, and 1.7 mph at the midpoint of the curve. Passenger car speeds decreased by only 0.4 mph at the midpoint after considering passenger cars entered the curve by approximately 1.3 mph slower in the first after period. For heavy trucks, there was a statistically significant reduction in speed of 3.5 mph in the first after period at the PC.

Table 24 shows that mean speed for passenger cars significantly increased at the approach point by approximately 2.5 mph in the second after period compared with the first after period at the treatment site. Although not significant, the speed increase was approximately 1.2 mph at the approach point from the before period to the second after period. The significant increase in speed in the second after period continued through the PC location, where the increase was approximately 1.8 mph relative to the before period and 3.1 mph relative to the first after period. Passenger car speeds were significantly reduced in the second after period when compared with the before period at the comparison site, at the approach point and the PC, by approximately 1.3 mph and 1.4 mph, respectively. This reduction in speeds at the comparison site increases the significance of the increase in speeds at the approach point and PC at the treatment site. Passenger car mean speeds were significantly lower at the curve midpoint by approximately 2.2 mph, in the second after period, not taking into account the speed reduction at the comparison site. However, speeds were also significantly reduced at the midpoint of the comparison site by approximately 1.7 mph. When taking this reduction into account, the speed reduction at the curve midpoint of the treatment site was no longer statistically significant. As with the WV Route 32 site, the high-friction surface had no long-term effect on passenger car mean operating speeds at the curve midpoint of the U.S. Route 33 treatment site. However, there was a statistically significant speed reduction at both the PC and midpoint for trucks in the second after period.

Table 24. Operating speeds at U.S. Route 33 before and after treatment.

Speed Metric	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N
Treatment PCs before	47.608	6.538	39.544	5.346	32.367	4.571	158
Treatment PCs 1st after	**46.301**	7.512	**38.245**	5.771	**30.614**	3.791	163
Treatment PCs 2nd after	***48.769***	7.723	***41.336***	7.199	**<u>30.154</u>**	5.024	143
Treatment trucks before	34.158	7.762	32.368	3.7	27.263	4.188	19
Treatment trucks 1st after	33.50	11.329	**28.833**	8.855	27.167	4.967	24
Treatment trucks 2nd after	30.905	9.049	**27.476**	7.593	***24.048***	5.661	21
Comparison PCs before	43.661	5.812	39.347	6.318	36.169	6.786	118
Comparison PCs 1st after	42.865	5.372	38.361	5.055	35.459	6.086	133
Comparison PCs 2nd after	**42.366**	6.462	**37.93**	5.812	**34.443**	5.445	142
Comparison trucks before	39.214	5.409	35.286	5.045	32.00	5.698	14
Comparison trucks 1st after	36.621	8.918	32.897	8.461	31.517	7.41	29
Comparison trucks 2nd after	36.80	5.569	34.70	4.921	32.55	5.781	20

Bold indicates significance at the 95-percent confidence level (α=0.05) when comparing either after period to the before period.
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods.
Underline indicates that, when accounting for the speed change at the comparison site, the change in the speed at the treatment site was not statistically significant at the 95 percent confidence level (α = 0.05).
SD = Standard Deviation
PC = Point of Curvature
N = Number of Observations

Figure 49 and figure 50 show graphical representations of the speed profiles for the U.S. Route 33 treatment and comparison sites, respectively. The general shape of the speed profiles for the three collection periods remained relatively similar.

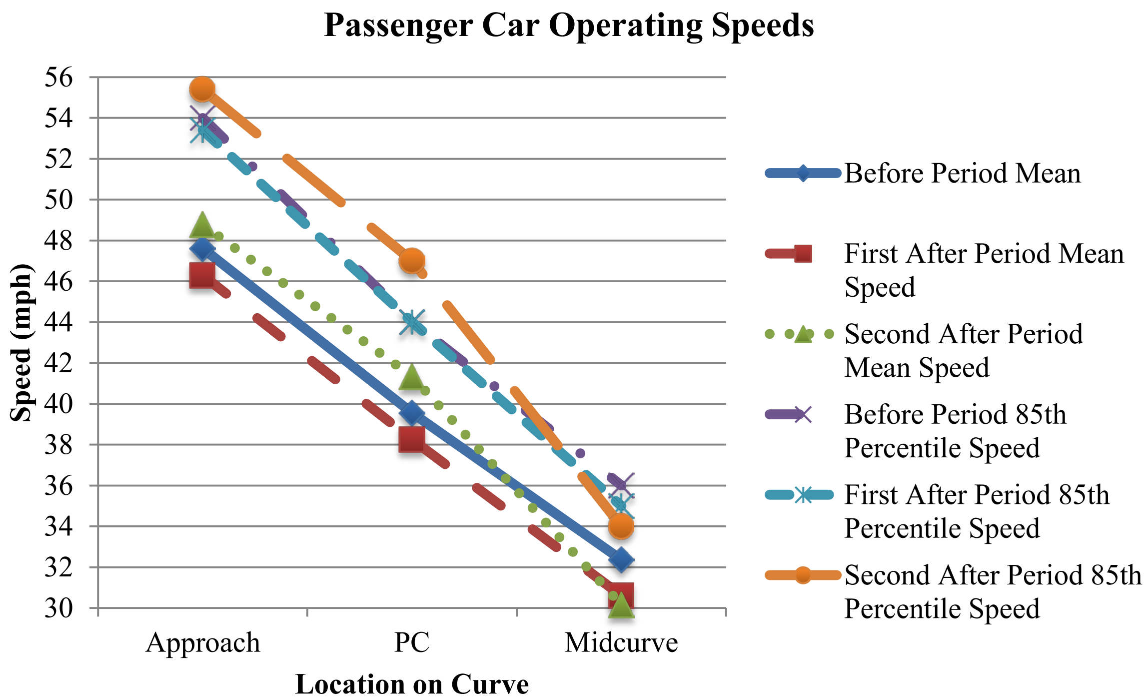


Figure 49. Graph. U.S. Route 33 treatment site operating speeds (PSL = 55 mph).

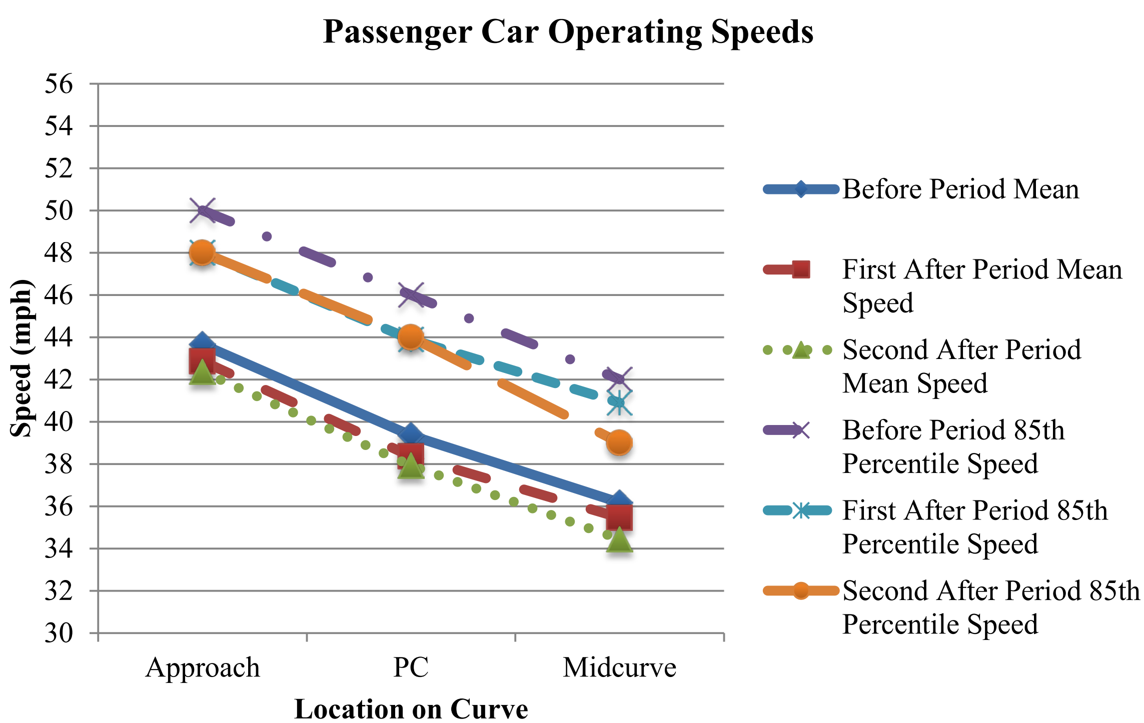


Figure 50. Graph. U.S. Route 33 comparison site operating speeds (PSL = 55 mph).

U.S. Route 219 (Treatment and Comparison Site)

Table 25 shows the mean speeds for passenger cars and heavy trucks for the treatment and comparison sites on U.S. Route 219. For these sites, there was a statistically significant increase in mean speed from the before period to the first after period at all three data-collection locations at the comparison site. This result was expected because the roadway was slightly damp in the before period. The data at the comparison site showed that approach and PC mean speeds increased by about 2.2 mph in the first after period relative to the before period. At the midcurve location, the mean speed increased by 2.7 mph when comparing the first after period with the before period. Without accounting for this change, the PC mean speed was significantly lower in the first after period relative to the before period at the MM 6.32 location, while the midpoint curve mean speed is significantly higher in the first after period relative to the before period. However, when taking the increase in speed at the comparison site into account, the speed increase at the curve midpoint of the first treatment site (MM 6.32) is no longer statistically significant. However, when accounting for the statistically significant reduction in the mean speed at the PC location, the PC mean speed reduction for passenger cars at the treatment site was approximately 4.8 mph, which was statistically significant. Passenger car speeds also increased, between the before period and first after period, at the second treatment site (MM 5.81), at the approach point and the midpoint of the curve. The speed increase was approximately 2.3 mph at both data-collection points. However, neither of these mean speed increases was statistically significant when considering the speed increases at the comparison site.

There were statistically significant changes at all three data-collection points at the comparison site in the second after period for passenger cars. There was a statistically significant increase in mean speed at the approach point between the before and second after periods, which was approximately 3.7 mph. There was also a statistically significant reduction in speed at the PC when comparing the second after period with the before period. There was no significant difference in speed at the midpoint of the curve between the second after and before periods. However, there was a statistically significant reduction in speed at the PC and curve midpoint between the first and second after periods. The speed reductions were approximately 4.6 mph at the PC and 2.7 mph at the curve midpoint.

Table 25 shows a significant speed reduction at the PC of the first treatment site (MM 6.32) in the second after period. This significant reduction was approximately 4 mph when compared with the before period and 1.4 mph when compared with the first after period. There was also a statistically significant speed increase at the curve midpoint of approximately 5.7 mph when compared with the before period, and 1.6 mph when compared with the first after period. Passenger cars maintained their speed from the PC to the curve midpoint at the first treatment site (MM 6.32), which suggests drivers may realize they have more available friction in the curve and do not decelerate as much throughout the curve. This is the only treatment site where speeds significantly increased in the curve, after taking into account speed changes at the comparison site, but this change was large, at approximately 5.7 mph. There also was a significant speed reduction at the approach point of the second treatment site (MM 5.81). This significant reduction was approximately 4 mph when compared with the first after period, but this speed reduction was not statistically significant when compared with the before period. There was a statistically significant speed reduction at the curve midpoint of approximately

3.4 mph when compared with the first after period, but again, this speed reduction was not significant when compared with the before period. This speed reduction was no longer statistically significant after accounting for the speed reduction at the comparison site.

There was no significant difference for heavy trucks from the before to first after period at the comparison site. There was a statistically significant increase in speed at the first treatment site (MM 6.32) at the curve midpoint of approximately 3.4 mph. However, the speed at the midpoint of the curve decreased in the second after period, and there was no significant change between the before period and second after period. The only significant change in truck speeds at this site in the second after period was at the PC, where there was a reduction of approximately 8.9 mph. Trucks then increased their mean speed by 0.6 mph to the curve midpoint, although this increase was not statistically significant. However, there were only five trucks in the second after period, which likely contributed to the large change in mean speed at the PC location. Of note, truck mean speeds at the comparison site in the second after period were significantly greater when compared with the before and first after periods. There was approximately a 12.6-mph increase at the approach, a 12.4-mph increase at the PC, and approximately a 13.2-mph increase at the curve midpoint. Trucks maintained relatively similar speeds throughout the three data-collection points in all three time periods at the comparison site, but the speeds were much greater in the second after periods. All three truck speeds (approach, PC, and curve midpoint) in the second after period were greater than passenger vehicles at the comparison site.

The only change in the second treatment site (MM 5.81) mean truck speed was between the first and second after periods, at the approach data-collection location. However, this reduction was not statistically significant when compared with the before period.

Table 25. Operating speeds at U.S. Route 219 before and after treatment.

Speed Metric	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N
Treatment 6.32 PCs before	48.221	6.282	46.465	6.135	36.291	7.221	86
Treatment 6.32 PCs 1st after	47.773	6.219	**43.876**	5.251	<u>**40.34**</u>	5.433	97
Treatment 6.32 PCs 2nd after	47.756	7.204	***42.511***	5.637	***41.967***	4.72	90
Treatment 6.32 trucks before	44.364	5.971	44.455	6.44	34.545	4.698	11
Treatment 6.32 trucks 1st after	45.063	5.118	40.75	6.017	**37.938**	5.092	16
Treatment 6.32 trucks 2nd after	42.00	7.517	**35.6**	5.413	36.20	4.919	5
Treatment 5.81 PCs before	48.696	8.825	45.544	7.412	34.797	6.728	79
Treatment 5.81 PCs 1st after	<u>**51.078**</u>	5.788	45.553	5.489	<u>**37.136**</u>	6.463	103
Treatment 5.81 PCs 2nd after	***47.042***	7.617	47.099	7.484	***33.718***	5.197	71
Treatment 5.81 trucks before	46.923	8.19	39.308	5.964	32.308	3.728	13
Treatment 5.81 trucks 1st after	50.566	3.395	40.667	4.444	32.778	4.466	9
Treatment 5.81 trucks 2nd after	46.00	2.828	44.5	7.778	31.50	4.95	2
Comparison PCs before	44.66	6.317	42.06	4.867	41.16	4.354	50
Comparison PCs 1st after	**46.831**	5.967	**44.282**	6.388	**43.859**	5.354	71
Comparison PCs 2nd after	**48.363**	7.915	***39.682***	11.257	***41.122***	5.802	11/41[a]
Comparison trucks before	41.429	8.08	40.286	5.908	39.857	4.1	7
Comparison trucks 1st after	39.143	10.04	38.143	8.591	37.857	7.925	7
Comparison trucks 2nd after	***54.00***	8.164	***52.636***	6.439	***53.091***	3.33	4/11[b]

[a]Eleven observations at the approach point and 41 observations at the PC and midcurve.
[b]Four observations at the approach point and 11 observations at the PC and midcurve.
Bold indicates significance at the 95-percent confidence level (α=0.05) when comparing either after period with the before period.
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods.
Underline indicates that, when accounting for the speed change at the comparison site, the change in the speed at the treatment site was not statistically significant at the 95-percent confidence level (α=0.05).
SD = Standard Deviation
PC = Point of Curvature
N = Number of Observations

Figure 51 through figure 53 show speed profiles for the U.S. Route 219 treatment and comparison sites. At the comparison site, the speed profiles show vehicles behaved in the same manner during the before and first after period, but in the second after period, vehicles had higher speeds at the approach point and then decelerated more aggressively to the PC, compared with the before and first after period. In the second after period, vehicles actually accelerated slightly from the PC to the curve midpoint, at the comparison site. At the first treatment site (MM 6.32), passenger cars had higher speeds at the approach point and PC, when compared with both after periods, but then they decelerated more aggressively from the PC to the curve midpoint and had a significantly lower speed at the curve midpoint compared with the after periods. Vehicles behaved relatively similarly between the two after periods, except the 85th percentile speed at the PC in the second after period was 2 mph less than in the first after period and 3 mph less than in the before period. At the second treatment site (MM 5.81), passenger cars behaved relatively similarly in the before and first after period. However, in the second after period, vehicles had slower speeds at the approach point and then slightly higher speeds at the PC; then they decelerated more aggressively to the curve midpoint and were traveling a at slower speed at the curve midpoint. This suggests drivers may be misjudging the curve, causing them to enter the curve at an excessive speed, and as they realized the tight geometry, causing them to brake harder in the curve.

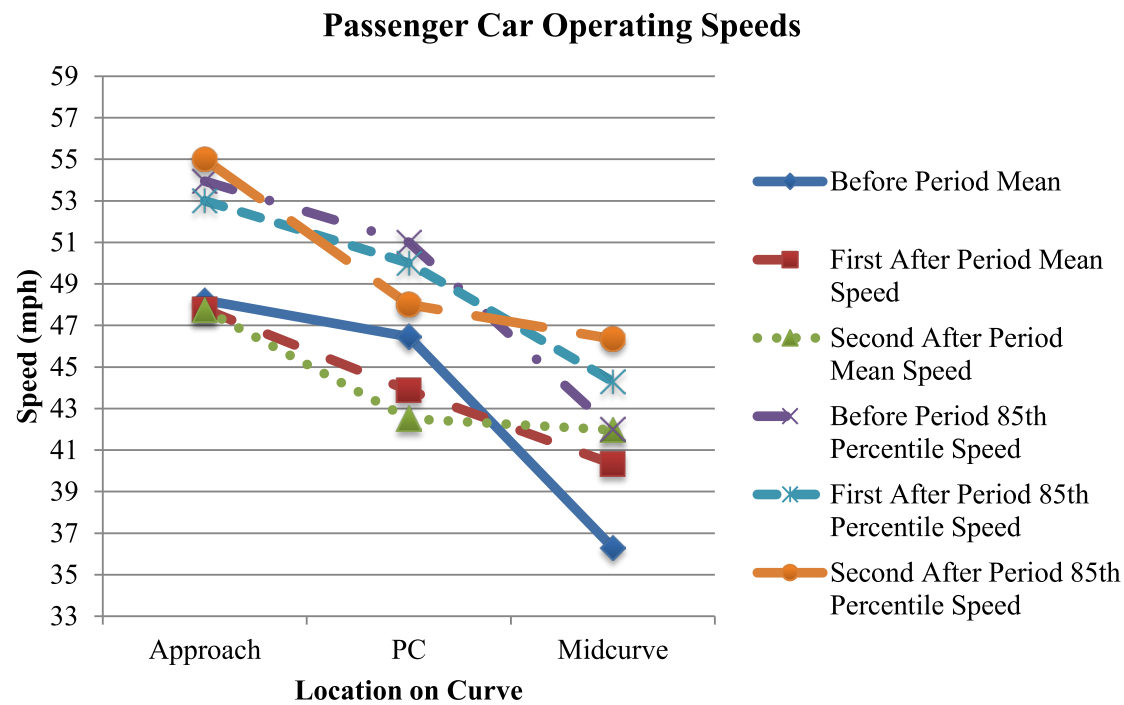


Figure 51. Graph. U.S. Route 219 treatment site A (MM 6.32) operating speeds (PSL = 55 mph).

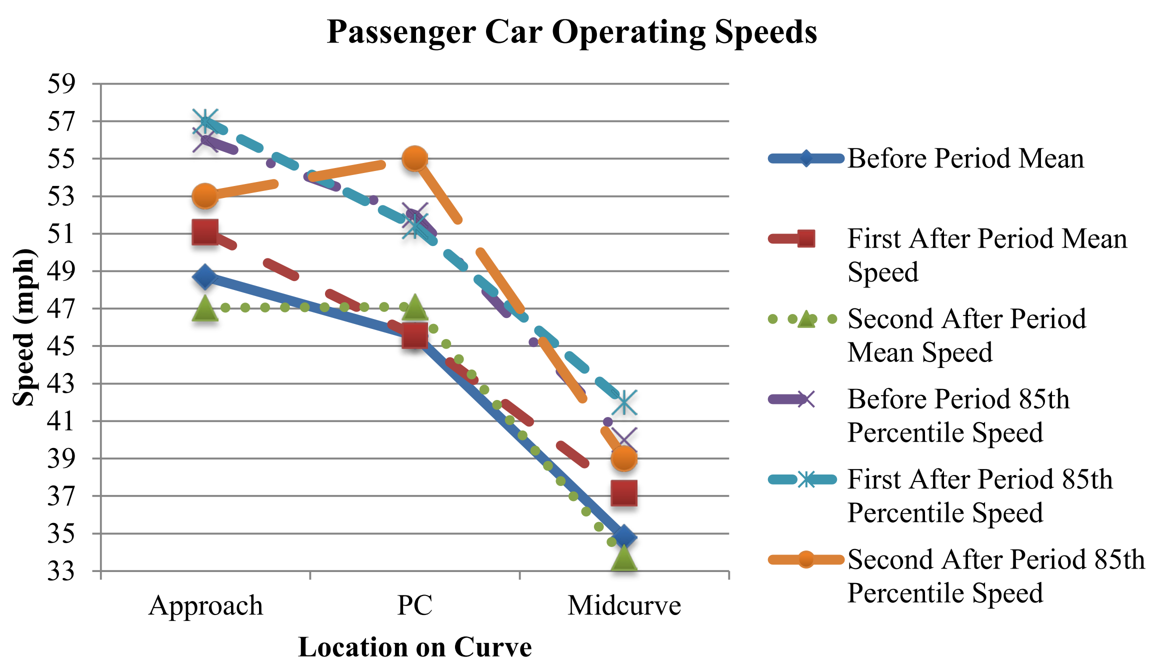


Figure 52. Graph. U.S. Route 219 treatment site B (MM 5.81) operating speeds (PSL=55 mph).

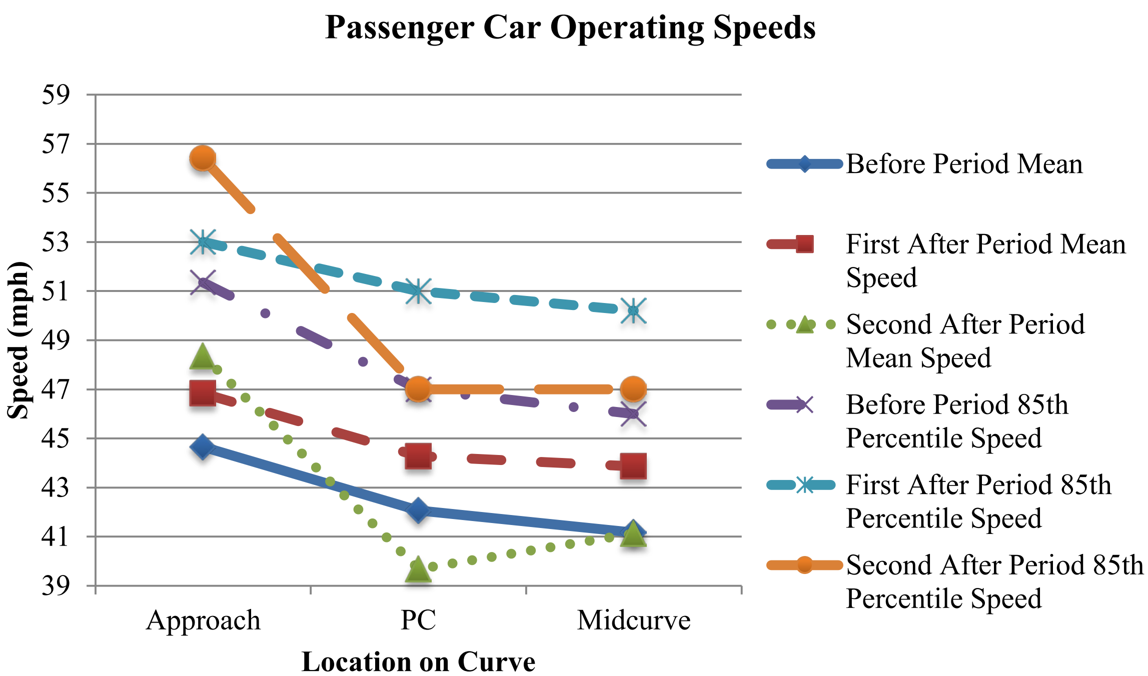


Figure 53. Graph. U.S. Route 219 comparison site operating speeds (PSL = 55 mph).

Mean Operating Speed Summary

In summary, the research team compared the mean operating speed for passenger cars and trucks before and at two different time periods after installing the HFST at four treatment and three comparison sites in West Virginia. The mean speeds were compared during the three data-collection time periods at a location on the approach tangent, curve PC location, and the midpoint of the horizontal curve. The analysis included only daytime and unopposed free-flow vehicle mean speeds.

The results of the mean speed analysis indicate that the HFST did not significantly affect vehicle mean operating speeds in a consistent manner across all four treatment sites. Few of the independent sample *t*-tests showed that the mean passenger car and heavy truck operating speeds differed between the before and first or second after periods at the treatment sites. A multifactor analysis of variance (ANOVA) found that the data collection-time period–treatment site interaction was not statistically significant ($F_{2,\,4160} = 0.49$, $p > 0.05$) at the approach speed location, when including observations of free-flow vehicles only and controlling for the presence of vehicles in the opposing lane, time of day, and vehicle type. Similar results were found at the PC ($F_{2,4160} = 1.51$, $p > 0.05$) and midcurve locations ($F_{2,\,4160} = 0.56$, $p > 0.05$).

Change in Mean Operating Speed

The next measure considered in the analysis was the change in speed from the PC to the midpoint of the horizontal curve (hereafter referred to as delta speed). This measure was observed in the before and both after periods at each of the treatment and comparison sites. As discussed previously, researchers used a *t*-test for independent samples to compare the two after periods with the before period and to compare the two after periods with one another. Table 26 shows the change in speed (delta speed) from the entry of the curve to the midpoint of the curve. The positive value of delta speed indicates that a mean decrease in speed occurred from the PC to the midpoint of the curve. A negative value represents a mean increase in speed between the two locations. As expected, few locations demonstrated a mean increase in speed from the PC to midcurve location.

Table 26. Change in speed from PC to curve midpoint.

Speed Metric	Delta Speed	Standard Deviation	N
U.S. Route 219 Treatment 6.32 PCs before	10.174	8.896	86
U.S. Route 219 Treatment 6.32 PCs 1st after	**3.536**	5.629	97
U.S. Route 219 Treatment 6.32 PCs 2nd after	***0.544***	4.913	90
U.S. Route 219 Treatment 6.32 trucks before	9.909	9.016	11
U.S. Route 219 Treatment 6.32 trucks 1st after	**2.813**	4.004	16
U.S. Route 219 Treatment 6.32 trucks 2nd after	**-0.6**	3.715	5
U.S. Route 219 Treatment 5.81 PCs before	10.747	7.923	79
U.S. Route 219 Treatment 5.81 PCs 1st after	**8.417**	7.239	103
U.S. Route 219 Treatment 5.81 PCs 2nd after	***13.38***	6.779	71
U.S. Route 219 Treatment 5.81 trucks before	7	7.165	13
U.S. Route 219 Treatment 5.81 trucks 1st after	7.889	1.691	9
U.S. Route 219 Treatment 5.81 trucks 2nd after	13	2.828	2
U.S. Route 219 Comparison PCs before	0.9	4.586	50
U.S. Route 219 Comparison PCs 1st after	0.423	5.008	71
U.S. Route 219 Comparison PCs 2nd after	-1.439	9.859	41
U.S. Route 219 Comparison trucks before	0.429	7.231	7
U.S. Route 219 Comparison trucks 1st after	0.286	1.976	7
U.S. Route 219 Comparison trucks 2nd after	-0.455	8.067	11
U.S. Route 33 Treatment PCs before	7.177	5.367	158
U.S. Route 33 Treatment PCs 1st after	7.632	4.953	163
U.S. Route 33 Treatment PCs 2nd after	***11.182***	7.653	143
U.S. Route 33 Treatment trucks before	5.105	3.71	19
U.S. Route 33 Treatment trucks 1st after	**1.667**	6.391	24
U.S. Route 33 Treatment trucks 2nd after	3.429	6.25	21
U.S. Route 33 Comparison PCs before	3.178	8.494	118
U.S. Route 33 Comparison PCs 1st after	2.902	6.393	133
U.S. Route 33 Comparison PCs 2nd after	3.486	5.312	142
U.S. Route 33 Comparison trucks before	3.286	8.09	14
U.S. Route 33 Comparison trucks 1st after	1.379	5.551	29
U.S. Route 33 Comparison trucks 2nd after	2.15	2.925	20
WV Route 32 Treatment PCs before	5.689	5.856	167
WV Route 32 Treatment PCs 1st after	5.783	7.091	175
WV Route 32 Treatment PCs 2nd after	5.216	7.072	199
WV Route 32 Treatment trucks before	1.667	4.179	6
WV Route 32 Treatment trucks 1st after	**8.875**	3.758	8
WV Route 32 Treatment trucks 2nd after	***2.917***	6.007	12
WV Route 32 Comparison PCs before	2.282	7.139	124
WV Route 32 Comparison PCs 1st after	2.357	4.548	185
WV Route 32 Comparison PCs 2nd after	2.933	4.97	240
WV Route 32 Comparison trucks before	4.429	4.117	7
WV Route 32 Comparison trucks 1st after	1.889	4.197	9
WV Route 32 Comparison trucks 1st after	2.688	5.474	16

Bold indicates significance at 95 percent confidence level (α=0.05) when comparing either after period with the before period.
Italics indicates significance at 95-percent confidence level (α=0.05) between the 1st and 2nd after periods.
N = Number of Observations

As table 26 shows, there was no statistically significant change in delta speed at any of the comparison locations for either passenger cars or heavy trucks. The only changes that occurred for passenger cars in the first after period were at the U.S. Route 219 treatment sites. In both cases, the delta speed was reduced (more constant speeds from the approach to the midpoint of the curve). Speeds became even more constant between the PC and midcurve in the second after treatment period at the first treatment site (MM 6.32) of U.S. Route 219, with less than a 1-mph change between the PC and curve midpoint. There was also a change at the second treatment site (MM 5.81) in the second after period; however, this change was an increase in delta speed of approximately 2.6 mph from the before period. This increase in delta speed is probably associated with passenger vehicles entering the curve at a higher speed in the second after period relative to the before period, and then realizing they are travelling too fast for the curve. There was also an increase in delta speed at the U.S. 33 treatment site in the second after period. The change was approximately 4 mph relative to the before period.

A statistically significant change was found for heavy trucks at three out of the four treatment sites in the first after period. The delta speed was significantly reduced at two sites (by 7.1 mph and 3.5 mph). One treatment site showed a significant increase in delta speed (7.2 mph) from the before to the first after period; however, it is noted that there were only six heavy vehicles in the before period and eight heavy vehicles in the after period. The very low SD in both cases for this site shows that outliers are not driving these results. However, delta speed was reduced by 6 mph from the first after period to the second after period, and was 1.3 mph greater than in the before period. The site with the 7.1 mph reduction in delta speed in the first after period also had a 3.4 mph reduction from the first after period to the second after period. Delta speed actually became slightly negative at this site, representing an increase in speed from the PC to the curve midpoint.

There is no consistent trend in the difference between passenger car speeds at the PC and curve midpoint from the before to the second after period as a result of the HFST. There was a significant increase in the difference at two treatment sites, but there was also a very significant decrease in the difference at another treatment site. The site (U.S. Route 219 Treatment 6.32) with the decrease in delta speed for passenger cars also had a statistically significant decrease in delta speed for trucks. A multifactor ANOVA found that the treatment site-time period interaction was not statistically significant ($F_{2,\,4160} = 0.51$, $p > 0.05$) when including observations of free-flow vehicles only and controlling for the presence of vehicles in the opposing lane, time of day, and vehicle type. This suggests that the HFST did not affect the change in vehicle speeds between the PC and midcurve locations in the current study.

Vehicles Exceeding Posted Speed Limit

Finally, in addition to the *t*-test, the percentage of vehicles exceeding the PSL and the advisory speed at the treatment and control sites was calculated and compared between data-collection periods. The following describes the results of the data analysis.

Table 27 shows the number of passenger cars and heavy trucks observed to exceed the speed limit at each location of every site in the before and both after periods. A test of proportions was used to determine whether the proportion of speeding vehicles changed at each location of every site from the before to after treatment application. From table 27, there were no visible trends from the before period to after treatment application. In the first after period, there was only a significant change in the number of passenger vehicles exceeding the speed limit at the first treatment site of U.S. Route 219 (MM 6.32), where the proportion of vehicles exceeding the speed limit at the PC was lower relative to the before period. Four vehicles exceeded the speed limit in the before period, and no vehicles exceeded the speed limit in the first and second after periods. This change was also significant in the second after period. There was one additional site in the second after period that had a significant reduction in the number of vehicles exceeding the speed limit. At the WV Route 32 treatment site, the proportion was reduced from 0.50 to 0.39 at the approach point. There were also a few locations where there was a significant difference between the two after periods, but not the before period, which resulted from a slight reduction in the first after period that was not statistically significant. The only statistically significant change at the curve midpoint was at the WV Route 32 comparison site, where the proportion of trucks exceeding the speed limit significantly increased in the first after period, but then the proportion decreased in the second after period and was no longer significant. This result is noteworthy, however, because the U.S. Route 219 and U.S. Route 33 sites had such restrictive geometry that few vehicles traveled in excess of the PSL.

Table 27. Number of vehicles exceeding the PSL by site.

Data-Collection Location and Time Period		Speed Observation Site						
		U.S. Route 219			U.S. Route 33		WV Route 32	
		Treat. A (MM 6.32)	Treat. B (MM 5.81)	Comp.	Treat.	Comp.	Treat.	Comp.
Approach passenger cars	Before	6 (7%)	15 (19%)	3 (6%)	15 (9%)	1 (1%)	83 (50%)	82 (66%)
	1st After	10 (10%)	23 (22%)	5 (7%)	12 (7%)	3 (2%)	75 (43%)	121 (65%)
	2nd After	12 (13%)	8 (11%)	1 (9%)	***21 (15%)***	3 (2%)	**77 (39%)**	143 (60%)
Approach trucks	Before	1 (9%)	2 (15%)	1 (14%)	0 (0%)	0 (0%)	4 (67%)	5 (71%)
	1st After	1 (6%)	1 (11%)	0 (0%)	0 (0%)	0 (0%)	5 (63%)	5 (56%)
	2nd After	0 (0%)	0 (0%)	***2 (50%)***	0 (0%)	0 (0%)	4 (33%)	5 31%)
PC passenger cars	Before	4 (5%)	5 (6%)	0 (0%)	0 (0%)	1 (1%)	63 (38%)	60 (48%)
	1st After	**0 (0%)**	2 (2%)	3 (4%)	3 (2%)	0 (0%)	52 (30%)	81 (44%)
	2nd After	**0 (0%)**	***8 (11%)***	3 (7%)	2 (1%)	0 (0%)	62 (31%)	125 (52%)
PC trucks	Before	1 (9%)	0 (0%)	0 (0%)	0 (0%)	0 (0%)	2 (33%)	1 (14%)
	1st After	0 (0%)	0 (0%)	0 (0%)	0 (0%)	0 (0%)	2 (25%)	3 (33%)
	2nd After	0 (0%)	0 (0%)	4 (36%)	0 (0%)	0 (0%)	3 (25%)	5 (31%)
Midcurve passenger cars	Before	1 (1%)	1 (1%)	0 (0%)	0 (0%)	2 (2%)	12 (7%)	45 (36%)
	1st After	1 (1%)	2 (2%)	0 (0%)	0 (0%)	0 (0%)	5 (3%)	48 (26%)
	2nd After	1 (1%)	0 (0%)	0 (0%)	0 (0%)	0 (0%)	***18 (9%)***	74 (31%)
Midcurve trucks	Before	0 (0%)	0 (0%)	0 (0%)	0 (0%)	0 (0%)	0 (0%)	**0 (0%)**
	1st After	0 (0%)	0 (0%)	0 (0%)	0 (0%)	0 (0%)	0 (0%)	**3 (33%)**
	2nd After	0 (0%)	0 (0%)	2 (18%)	0 (0%)	0 (0%)	0 (0%)	1 (6%)
Number of passenger cars	Before	86	79	50	158	118	167	124
	1st After	97	103	71	163	133	175	185
	2nd After	90	71	11/41[a]	143	142	199	240
Number of trucks	Before	11	13	7	19	14	6	7
	1st After	16	9	7	24	29	8	9
	2nd After	5	2	4/11[b]	21	20	12	16

[a]Eleven observations at the approach point and 41 observations at the PC and midcurve.
[b]Four observations at the approach point and 11 observations at the PC and midcurve.
Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period.
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods.
Treat. = Treatment
MM = Mile Marker
Comp. = Comparison
PC = Point of Curvature

Table 28 shows the number of vehicles exceeding the advisory speed at the midpoint of the curve for each study site. The research team considered only the midpoint location because the advisory speed is for the horizontal curve, and the approach and PC locations are not within the curve. No significant changes were observed at the comparison sites from the before to the first after period. The proportion of passenger cars exceeding the advisory speed was significantly higher at the U.S. Route 219 treatment A (MM 6.32) site, but there was no change at any other site in the first after period. This increase continued into the second after period, where all but one vehicle exceeded the advisory speed. The proportion of heavy trucks exceeding the advisory speed was significantly lower at the WV Route 32 site in the first after period, but there was no change at any other site. This reduction did not continue into the second after period and the proportion of trucks exceeding the advisory speed was 100 percent, which is the same as in the before period. In the second after period at the U.S. Route 219 treatment B (MM 5.81) site, there was a significant decrease in the proportion of vehicles exceeding the advisory speed when compared with the before and first after periods. At this same site, the proportion of trucks exceeding the advisory speed was also significantly reduced when compared with the before and first after periods. There was also a significant reduction at the U.S. Route 219 comparison site between the first after period and second after period. This reduction was less significant than at the treatment B site. The proportion of vehicles decreased by 0.08 compared with the before period at the comparison site and 0.19 at the treatment site B. The proportion of passenger cars exceeding the advisory speed increased at one treatment site, decreased at another treatment site, and remained the same at the other two. These results indicate there might be a slight reduction in the number of trucks exceeding the advisory speed, but there does not appear to be a strong effect on the number of passenger vehicles exceeding the advisory speed at the HFST locations.

Table 28. Number of vehicles exceeding the advisory speed by site.

		Speed Observation Site						
		U.S. Route 219			**U.S. Route 33**		**WV Route 32**	
Data-Collection Location and Time Period		**Treat. A (MM 6.32)**	**Treat. B (MM 5.81)**	**Comp.**	**Treat.**	**Comp.**	**Treat.**	**Comp.**
Midcurve passenger cars	Before	74 (86%)	75 (95%)	49 (98%)	148 (94%)	111 (94%)	154 (92%)	80 (65%)
	1st After	**94 (97%)**	100 (97%)	70 (99%)	148 (91%)	124 (93%)	159 (91%)	112 (61%)
	2nd After	**89 (99%)**	***54 (76%)***	***37 (90%)***	129 (90%)	134 (94%)	177 (89%)	155 (65%)
Midcurve trucks	Before	8 (73%)	13 (100%)	7 (100%)	13 (68%)	12 (86%)	6 (100%)	3 (43%)
	1st After	15 (94%)	9 (100%)	5 (71%)	16 (67%)	22 (76%)	**4 (50%)**	4 (44%)
	2nd After	4 (80%)	***1 (50%)***	11 (100%)	9 (43%)	19 (95%)	***12 (100%)***	6 (38%)
Number of passenger cars	Before	86	79	50	158	118	167	124
	1st After	97	103	71	163	133	175	185
	2nd After	90	71	41	143	142	199	240
Number of trucks	Before	11	13	7	19	14	6	7
	1st After	16	9	7	24	29	8	9
	2nd After	5	2	11	21	20	12	16

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st or 2nd after periods and the before period.
Italics indicates significance at 95 percent confidence level (α=0.05) between the 1st and 2nd after periods.
Treat. = Treatment
MM = Mile Marker
Comp. = Comparison
PC = Point of Curvature

Speed Variance Analysis

Speed variance was the final speed performance metric considered in the current study. A two-sided *F*-test was used to compare the variance of vehicle operating speed in the before and after periods for passenger cars and heavy trucks.

Table 29 shows that the only change that occurred for the U.S. Route 219 comparison site in the first after period was at the PC for passenger cars. There was an approximate 1.5-mph increase in speed deviation at this location. Taking this change into account, the speed deviation was found to decrease at the PC locations for both treatment sites. There was also a significant decrease at the approach location for treatment site B for both passenger cars and heavy trucks, as well as at the midpoint of the curve at treatment site A for passenger cars. For the U.S. Route 219 comparison site, there was an increase in speed deviation at the PC and curve midpoint in the second after period compared with the before period. There was an approximate 6.4-mph increase in speed deviation at the PC and an approximate 1.5-mph increase at the curve midpoint. Taking these changes into account, there was a decrease in speed deviation at the PC and curve midpoint for both treatment sites. There was only a significant decrease between the first and second after period for trucks at the midpoint of the curve of the comparison site. However, there was no change between the before and second after period. There was also no change in speed deviation at any data-collection point for trucks at the two treatment sites.

When comparing the first after period with the before period for U.S. Route 33 comparison site, the speed deviation decreased by 1.3 mph at the PC for passenger cars and increased by approximately 3.5 mph at the approach and PC for heavy trucks. Taking these trends into account, a significant increase in speed deviation occurred at the approach and PC for passenger cars, while a significant decrease occurred at the midpoint of the horizontal curve. For heavy trucks, a significant increase occurred only at the PC (approximately 2.7 mph). At the comparison site, the speed deviation decreased by approximately 1.3 mph at the curve midpoint for passenger cars. There was also a statistically significant increase at the approach for passenger cars, but this was only between the first and second after periods. Taking these trends into account, a significant increase in speed deviation occurred at the curve midpoint for passenger cars in the second after period. At the comparison site, there was no statistically significant change in speed deviation between the before and second after period for trucks. There was a statistically significant decrease at the approach and PC location for trucks between the first and second after periods. The speed deviation increased by approximately 3.9 mph at the PC for trucks from the before to second after period at the treatment site.

For WV Route 32, there were statistically significant decreases at the PC and the midpoint of the curve for passenger cars and significant increases at the approach and PC for heavy trucks at the comparison site in the first after period. Taking these differences into account resulted in statistically significant increases in speed deviation at the PC and midpoint for passenger cars, and significant decreases for heavy trucks at the approach and PC for heavy trucks at the treatment site. In the second after period, there was a significant increase in speed deviation at the PC and a significant decrease at the curve midpoint for the comparison site for passenger cars. Taking these differences into account, there was a significant increase in speed deviation at the approach and curve midpoint of the treatment site for passenger cars in the second after period. There was also a significant increase in speed deviation at the approach and PC for trucks at the comparison site. Taking these differences into account, there was a significant decrease in speed deviation at the approach and PC for the treatment site for trucks in the second after period.

There was no consistent trend in the effects of the HFST on the speed deviation of passenger cars. The effects of the HFST on the speed deviation were considered for the PC and curve midpoint locations because the surface was applied starting at the PC, or just before it, and it was assumed that the surface would have no effect on the speed deviation at the approach data-collection location. In the first after period, the speed deviation increased at the PC for two sites, but the speed deviation also decreased at two other sites. In the second after period, the speed deviation decreased at the PC for two sites. In the first after period, the speed deviation increased at the curve midpoint at one site, but the speed deviation decreased at another site. In the second after period, the speed deviation increased at the curve midpoint for two sites, but the speed deviation also decreased at two sites. For trucks, the speed deviation decreased at the PC at one site in both after periods, and it also decreased at one site in both after periods. There was no change in speed deviation at the curve midpoint for trucks at any site.

Table 29. Change in standard deviation of speed.

Speed Metric	Approach SD	PC SD	Mid SD	N
U.S. Route 219 Treatment 6.32 PCs before	6.282	6.135	7.221	86
U.S. Route 219 Treatment 6.32 PCs 1st after	6.219	5.251	**5.433**	97
U.S. Route 219 Treatment 6.32 PCs 2nd after	7.204	5.637	**4.72**	90
U.S. Route 219 Treatment 6.32 trucks before	5.971	6.44	4.698	11
U.S. Route 219 Treatment 6.32 trucks 1st after	5.118	6.017	5.092	16
U.S. Route 219 Treatment 6.32 trucks 2nd after	7.517	5.413	4.919	5
U.S. Route 219 Treatment 5.81 PCs before	8.825	7.412	6.728	79
U.S. Route 219 Treatment 5.81 PCs 1st after	**5.788**	**5.489**	6.463	103
U.S. Route 219 Treatment 5.81 PCs 2nd after	***7.617***	***7.484***	***5.197***	71
U.S. Route 219 Treatment 5.81 trucks before	8.19	5.964	3.728	13
U.S. Route 219 Treatment 5.81 trucks 1st after	**3.395**	4.444	4.466	9
U.S. Route 219 Treatment 5.81 trucks 2nd after	2.828	7.778	4.95	2
U.S. Route 219 Comparison PCs before	6.317	4.867	4.354	50
U.S. Route 219 Comparison PCs 1st after	5.967	**6.388**	5.354	71
U.S. Route 219 Comparison PCs 2nd after	7.915	***11.257***	**5.802**	11/41[a]
U.S. Route 219 Comparison trucks before	8.08	5.908	4.10	7
U.S. Route 219 Comparison trucks 1st after	10.04	8.591	7.925	7
U.S. Route 219 Comparison trucks 2nd after	8.164	6.439	***3.33***	4/11[b]
U.S. Route 33 Treatment PCs before	6.538	5.346	4.571	158
U.S. Route 33 Treatment PCs 1st after	**7.512**	5.771	**3.791**	163
U.S. Route 33 Treatment PCs 2nd after	**7.723**	***7.199***	***5.024***	143
U.S. Route 33 Treatment trucks before	7.762	3.7	4.188	19
U.S. Route 33 Treatment trucks 1st after	11.329	**8.855**	4.967	24
U.S. Route 33 Treatment trucks 2nd after	9.049	**7.593**	5.661	21
U.S. Route 33 Comparison PCs before	5.812	6.318	6.786	118
U.S. Route 33 Comparison PCs 1st after	5.372	**5.055**	6.086	133
U.S. Route 33 Comparison PCs 2nd after	***6.462***	5.812	**5.445**	142
U.S. Route 33 Comparison trucks before	5.409	5.045	5.698	14
U.S. Route 33 Comparison trucks 1st after	**8.918**	**8.461**	7.41	29
U.S. Route 33 Comparison trucks 2nd after	***5.569***	***4.921***	5.781	20
WV Route 32 Treatment PCs before	7.3	6.283	5.639	167
WV Route 32 Treatment PCs 1st after	7.959	**7.343**	5.126	175
WV Route 32 Treatment PCs 2nd after	**8.447**	6.645	***7.212***	199
WV Route 32 Treatment trucks before	3.987	5.089	4.622	6
WV Route 32 Treatment trucks 1st after	3.227	5.855	4.658	8
WV Route 32 Treatment trucks 2nd after	***6.443***	5.384	3.477	12

Speed Metric	**Approach SD**	**PC SD**	**Mid SD**	**N**
WV Route 32 Comparison PCs before	6.04	6.046	7.617	124
WV Route 32 Comparison PCs 1st after	5.317	**5.057**	**5.587**	185
WV Route 32 Comparison PCs 2nd after	*6.315*	***6.923***	**6.06**	240
WV Route 32 Comparison trucks before	2.992	2.734	3.823	7
WV Route 32 Comparison trucks 1st after	**8.023**	**5.94**	5.148	9
WV Route 32 Comparison trucks 2nd after	**6.811**	**6.708**	3.373	16

[a]Eleven observations at the approach point and 41 observations at the PC and midcurve.
[b]Four observations at the approach point and 11 observations at the PC and midcurve.
Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods relative to the before period.
Italics indicates significance at 95-percent confidence level (α=0.05) between the 1st and 2nd after periods.
SD = Standard Deviation
PC = Point of Curvature
N= Number of Observations

Encroachment Data

Along with speed metrics, the research team observed encroachment and braking vehicles; table 30 shows these data. The data collection and analysis methods were described earlier in this report.

Table 30 shows the braking and encroachment data for the comparison and treatment sites. There was no statistically significant change in the before or first after period—for any braking or encroachment measure—at treatment site A along U.S. Route 219. At treatment site B along U.S. Route 219, there was a statistically significant reduction in the proportion of vehicles entering the curve that encroached onto the shoulder and those vehicles entering the curve that encroached over the centerline, when comparing the before with the first after period. However, these reductions did not continue into the second after period, and there were no statistically significant change in the second after period relative to the before period. At the U.S. Route 219 comparison site, there was a reduction in the proportion of vehicles entering the curve that encroached onto the shoulder in the second after period. There was also a reduction in the proportion of vehicles entering the curve that braked inside the curve and a reduction in the proportion of vehicles exiting the curve that encroached beyond centerline, when comparing the before with the first after period. However, these reductions did not continue into the second after period, and there were no reductions in the second after period relative to the before period.

There was a reduction in the proportion of vehicles entering the curve that encroached on the centerline in the first after period at the U.S. Route 33 treatment site, but there was no reduction in the second after period. At both the U.S. Route 33 treatment and comparison sites, there was a reduction in the proportion of vehicles entering the curve that braked before the curve in the first after period, but there was no reduction in the second after period, relative to the before period. At the treatment site, there was a statistically significant increase in the proportion of vehicles exiting the curve that encroached onto the shoulder, in both after periods. There was also a significant reduction in the proportion of vehicles exiting the curve that crossed the centerline between the two lanes in the first after period, but once again, this reduction did not continue into the second after period.

At the WV Route 32 treatment site, there was an increase in the proportion of vehicles entering the curve that braked inside the curve in the first after period, but there was no change in the second after period relative to the before period. At the WV Route 32 treatment site, the camera was placed so it could see before the curve in the second after period, where it could not in the before and first after period. As a result, a statistical analysis of the braking before/within the curve for this site was not conducted in the second after period. There was also a reduction in the proportion of vehicles entering the curve that encroached onto the shoulder in the second after period at the treatment site. This significant reduction becomes more significant after accounting for the comparison site because there was an increase in the proportion of vehicles entering the curve that encroached onto the shoulder. Finally, at the WV Route 32 comparison site, there was a reduction in the proportion of vehicles entering the curve that braked before the curve in the first after period, but there was an increase in the second after period.

There was no significant change in the braking and encroachment data that lasted throughout the treatment period except for an increase in the proportion of vehicles exiting the curve that encroached onto the shoulder at the U.S. Route 33 treatment site and also a decrease in the proportion of vehicles entering the curve that encroached onto the shoulder at the WV Route 32 treatment site. These results show the HFST does not have a consistent effect on the braking and encroachment behavior of vehicles.

Table 30. Number of encroachments and braking vehicles by site.

Data-Collection Location and Time Period		Observation Site						
		U.S. Route 219			U.S. Route 33		WV Route 32	
		Treat. A (MM 6.32)	Treat. B (MM 5.81)	Comp.	Treat.	Comp.	Treat.	Comp.
Total entering	Before	77	61	49	122	89	127	110
	1st after	91	74	111	104	80	102	122
	2nd after	89	77	39	103	81	74	68
Entering shoulder	Before	2 (3%)	10 (16%)	8 (16%)	6 (5%)	4 (4%)	17 (13%)	2 (2%)
	1st after	3 (3%)	**0 (0%)**	12 (11%)	4 (4%)	4 (5%)	11 (11%)	3 (2%)
	2nd after	8 (9%)	***5 (6%)***	**1 (3%)**	11 (11%)	7 (9%)	***1 (1%)***	***8 (12%)***
Entering centerline	Before	0 (0%)	10 (16%)	0 (0%)	15 (12%)	10 (11%)	0 (0%)	0 (0%)
	1st after	1 (1%)	**4 (5%)**	0 (0%)	**5 (5%)**	8 (10%)	1 (1%)	0 (0%)
	2nd after	3	10	0 (0%)	8 (8%)	4 (5%)	3 (4%)	0 (0%)
Entering brake before	Before	56 (73%)	31 (51%)		115 (94%)	85 (96%)		15 (14%)
	1st after	75 (82%)	43 (58%)	N/A	**87 (84%)**	**62 (78%)**	N/A	**6 (5%)**
	2nd after	70 (79%)	50 (65%)	N/A	***96 (93%)***	***79 (98%)***	16 (22%)	***18 (26%)***
Entering brake inside	Before	12 (16%)	20 (33%)	47 (96%)	3 (2%)	2 (2%)	35 (28%)	10 (9%)
	1st after	6 (7%)	27 (36%)	**83 (75%)**	4 (4%)	**10 (13%)**	**53 (52%)**	17 (14%)
	2nd after	7 (8%)	23 (30%)	***38 (97%)***	5 (5%)	***1 (1%)***	23 (31%)	6 (9%)
Total exiting	Before	92	61	60	130	92	132	129
	1st after	69	62	78	124	93	96	97
	2nd after	90	71	33	94	100	88	76
Exiting shoulder	Before	3 (3%)	2 (3%)	3 (5%)	1 (1%)	12 (13%)	6 (5%)	0 (0%)
	1st after	3 (4%)	1 (2%)	1 (1%)	**9 (7%)**	5 (5%)	5 (5%)	0 (0%)
	2nd after	2 (2%)	2 (3%)	0 (0%)	***18 (19%)***	9 (9%)	2 (2%)	1 (1%)
Exiting centerline	Before	6 (7%)	12 (20%)	20 (33%)	2 (2%) [48] (37%)	5 (5%)	12 (9%)	6 (5%)
	1st after	3 (4%)	5 (8%)	**11 (14%)**	0 (0%) **[20](16%)**	3 (3%)	7 (7%)	4 (4%)
	2nd after	10 (11%)	***16 (23%)***	***12 (36%)***	0 (0%) [24] (26%)	4 (4%)	7 (8%)	3 (4%)

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period.
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods.
Brackets indicate encroachment on broken line for U.S. 33 treatment site in uphill (exiting) direction
Treat. = Treatment
MM = Mile Marker
Comp. = Comparison

Friction Data

The research team used the DF tester and CT meter to evaluate the skid resistance of the treatment and comparison site locations. The data collection and analysis methods were described earlier in this report. Table 31 through table 33 show the before, first after, and second after period friction data, respectively. The values reported in each table correspond to the SN at 40 mph. As table 31 shows, the minimum friction value observed in the before period was 0.26, and the maximum friction value was 0.46. As noted above, the research team observed friction values across the curve approach to determine whether a trend existed. As the data show, there was no discernible trend across the approaches (where deceleration occurs). The data do show that there is not a consistent level of friction across each approach, but each location varies with respect to its adjacent location differently at each site. As for the horizontal curve, there is a large amount of variability in pavement friction within each horizontal curve in the before period. At the WV Route 32 and U.S. Route 33 sites, the minimum friction level occurred at the three-quarter point of the horizontal curve; however, this was not the case at the U.S. Route 219 sites. At the U.S. Route 219 sites, the minimum friction level occurred at the one-quarter point or the midpoint of the horizontal curve. Otherwise, there are no apparent patterns within the horizontal curves with regard to friction supply.

Table 31. SN65 friction data from the before period.

Site	300 ft	200 ft	100 ft	PC	1/4	Midcurve	3/4	PT
WV Route 32 Treatment	0.41	0.46	0.40	0.39	0.32	0.38	0.31	0.32
WV Route 32 Comparison	0.38	0.41	0.40	0.36	0.36	0.36	0.28	0.42
U.S. Route 33 Treatment	0.33	0.34	0.34	0.40	0.33	0.32	0.32	0.36
U.S. Route 33 Comparison	N/A	N/A	0.31	0.34	0.28	0.28	0.26	0.29
U.S. Route 219 Treatment MM 6.32	0.38	0.37	0.36	0.40	0.36	0.38	0.37	0.40
U.S. Route 219 Treatment MM 5.81	0.38	0.40	0.40	0.41	0.40	0.35	0.44	0.40
U.S. Route 219 Comparison	N/A	N/A	0.31	0.32	0.29	0.30	0.33	0.32

MM = Mile Marker
PT = Point of Tangent
N/A = Not Applicable

Table 32 presents the friction data from the immediate after-application period. The HFST was only applied within the horizontal curves (from the PC to the PT) at each treatment site. This means that the approach data at every site and all data at the comparison sites should be comparable. The treatment surface was applied from the PC to the PT for each of the treatment curves. For the WV Route 32 treatment site, it is readily apparent that the treatment application was not uniform—the observed friction decreased from the PC to the PT. At the other three sites, the application appeared to provide the greatest observed friction at the PC and PT; the friction across the middle of the curve was lower than at the ends of the curve.

Table 32. SN65 friction data from the immediate after period.

Site	300 ft	200 ft	100 ft	PC	1/4	Midcurve	3/4	PT
WV Route 32 Treatment	0.31	0.42	0.43	0.72	0.67	0.66	0.62	0.57
WV Route 32 Comparison	0.32	0.32	0.37	0.35	0.33	0.33	0.35	0.38
U.S. Route 33 Treatment	0.36	0.32	0.38	0.62	0.60	0.51	0.56	0.58
U.S. Route 33 Comparison	N/A	N/A	0.25	0.27	0.26	0.29	0.26	0.30
U.S. Route 219 Treatment MM 6.32	0.35	0.32	0.29	0.69	0.56	0.56	0.58	0.60
U.S. Route 219 Treatment MM 5.81	0.37	0.38	0.34	0.65	0.61	0.62	0.60	0.66
U.S. Route 219 Comparison	N/A	N/A	0.30	0.31	0.26	0.29	0.27	0.34

MM = Mile Marker
PT = Point of Tangent
N/A = Not Applicable

Table 33 shows the friction data from the second after period. As mentioned in the immediate after period at WV Route 32, the observed friction decreased from the PC to PT, with the exception that the observed friction at the PT was almost as high as that at the PC location. The friction levels within the horizontal curve decreased between 0.12 and 0.15 at the WV Route 32 treatment site from the immediate after period to the second after period, with the exception of the PT location, which increased by 0.1. As mentioned in the immediate after period discussion, it is apparent that the treatment was not uniform throughout the curve. The coefficient of friction decreased between 0.10 and 0.15 at the U.S. Route 33 treatment site from the immediate after period to the second after period. The coefficient of friction values at the U.S. Route 219 treatment sites did not change as much between the two after periods as they did at the U.S. Route 33 and WV Route 32 treatment sites.

Table 33. SN65 friction data from the second after period.

Site	300 ft	200 ft	100 ft	PC	1/4	Midcurve	3/4	PT
WV Route 32 Treatment	0.36	0.38	0.36	0.59	0.55	0.51	0.50	0.58
WV Route 32 Comparison	0.43	0.36	0.35	0.37	0.29	0.28	0.30	0.35
U.S. Route 33 Treatment	0.32	0.34	0.34	0.51	0.46	0.38	0.46	0.43
U.S. Route 33 Comparison	N/A	N/A	0.34	0.29	0.29	0.29	0.29	0.30
U.S. Route 219 Treatment MM 6.32	0.33	0.36	0.34	0.59	0.58	0.59	0.61	0.61
U.S. Route 219 Treatment MM 5.81	0.39	0.40	0.43	0.61	0.55	0.51	0.59	0.59
U.S. Route 219 Comparison	N/A	N/A	0.33	0.37	0.29	0.32	0.36	0.36

MM = Mile Marker
PT = Point of Tangent
N/A = Not Applicable

Table 34 presents the before and after friction data for the treatment site approaches and curves, and also presents the data for the comparison sites. The research team conducted a *t*-test to compare the before and both after periods of friction data at each site. The test was statistically significant (p-value < 0.05) at each of the four treatment curves in the first after period relative to the before period. The friction level increased significantly from the before to the after period at the treatment sites, ranging from an increase of 0.21 to 0.30 in the SN65 value. At all four treatment sites, the second after period produced friction levels that were statistically higher than the before period, indicating that the higher friction levels produced by the treatment were maintained for at least one year.

Only two sites had friction values that were not significantly different from the before to the after period. The first was the approach to the WV Route 32 treatment site and the second was the U.S. 219 comparison site. This means that for the rest of the treatment approaches and comparison sites (none of which were treated with a high-friction surface), the before and after friction cannot be treated as the same value. The differences were marginal, as can be noted from table 34, but may indicate differences resulting from weather conditions or some other confounding factor from the before period (in May) to the after period (in August). Because of these findings, the before and after friction data at the comparison and non-treatment sites were considered independently in the margin of safety analysis.

Table 34. Before and after friction data for comparison.

Location	Mean	Standard Deviation	N	*t*-stat
WV Route 32 Treatment Approach before	0.424	0.032	6	N/A
WV Route 32 Treatment Approach 1st after	0.386	0.068	6	-1.24
WV Route 32 Treatment Approach 2nd after	0.365	0.015	6	**-4.14**
WV Route 32 Treatment Curve before	0.347	0.037	10	N/A
WV Route 32 Treatment Curve 1st after	0.646	0.057	10	**13.90**
WV Route 32 Treatment Curve 2nd after	0.543	0.040	10	**11.35**
WV Route 32 Comparison Site before	0.370	0.044	16	N/A
WV Route 32 Comparison Site 1st after	0.344	0.026	16	**-2.04**
WV Route 32 Comparison Site 2nd after	0.341	0.050	16	-1.76
U.S. Route 33 Treatment Approach before	0.340	0.012	6	N/A
U.S. Route 33 Treatment Approach 1st after	0.369	0.017	6	**3.40**
U.S. Route 33 Treatment Approach 2nd after	0.336	0.010	6	-0.56
U.S. Route 33 Treatment Curve before	0.347	0.037	10	N/A
U.S. Route 33 Treatment Curve 1st after	0.576	0.043	10	**12.74**
U.S. Route 33 Treatment Curve 2nd after	0.447	0.048	10	**5.23**
U.S. Route 33 Comparison Site before	0.293	0.027	12	N/A
U.S. Route 33 Comparison Site 1st after	0.276	0.020	12	-1.75
U.S. Route 33 Comparison Site 2nd after	0.301	0.020	12	0.86
U.S. Route 219 Treatment 6.32 Approach before	0.372	0.018	6	N/A
U.S. Route 219 Treatment 6.32 Approach 1st after	0.318	0.029	6	**-3.87**
U.S. Route 219 Treatment 6.32 Approach 2nd after	0.343	0.012	6	**-3.34**
U.S. Route 219 Treatment 6.32 Curve before	0.381	0.030	10	N/A
U.S. Route 219 Treatment 6.32 Curve 1st after	0.599	0.053	10	**11.32**
U.S. Route 219 Treatment 6.32 Curve 2nd after	0.596	0.013	10	**21.12**
U.S. Route 219 Treatment 5.81 Approach before	0.394	0.017	6	N/A
U.S. Route 219 Treatment 5.81 Approach 1st after	0.363	0.032	6	**-2.09**
U.S. Route 219 Treatment 5.81 Approach 2nd after	0.406	0.021	6	1.11
U.S. Route 219 Treatment 5.81 Curve before	0.402	0.030	10	N/A
U.S. Route 219 Treatment 5.81 Curve 1st after	0.631	0.028	10	**17.62**
U.S. Route 219 Treatment 5.81 Curve 2nd after	0.570	0.040	10	**10.63**
U.S. Route 219 Comparison Site before	0.312	0.013	12	N/A
U.S. Route 219 Comparison Site 1st after	0.296	0.030	12	-1.70
U.S. Route 219 Comparison Site 2nd after	0.338	0.031	12	**2.70**

Bold indicates that the difference between the after and before period was statistically significant with a 95-percent confidence.
N = Number of Observations
N/A = Not Applicable

As table 34 shows, the friction levels decreased throughout the horizontal curves at three treatment sites and remained the same, from the first to second after period, at these same sites. The decreases ranged from 0.061 to 0.129. However, the coefficients of friction were all significantly higher in the second after period than in the before period. *t*-statistics ranged from 5.23 to 21.12. Friction levels at the comparison sites actually increased slightly at two sites and remained the same at the other site.

The friction data shown in table 35, table 36, and table 37 are longitudinal friction values for the sites. For the margin of safety analysis, the research team estimated the available side (lateral) friction. Lamm et al. has shown that available side (lateral) friction is approximately 92.5 percent of the available longitudinal friction.[74] The authors showed that a multiplier of 0.925 can be used to determine available side (lateral) friction from available longitudinal friction. Table 35, table 36, and table 37show that using this scaling factor, the estimated available side friction (at 40 mph) for the before and both after periods, respectively.

Table 35. SN65 estimated available side friction before treatment application.

Site	300 ft	200 ft	100 ft	PC	1/4	Midcurve	3/4	PT
WV Route 32 Treatment	0.38	0.43	0.37	0.36	0.30	0.35	0.29	0.30
WV Route 32 Comparison	0.35	0.38	0.37	0.33	0.33	0.33	0.26	0.39
U.S. Route 33 Treatment	0.31	0.31	0.31	0.37	0.31	0.30	0.30	0.33
U.S. Route 33 Comparison	N/A	N/A	0.29	0.31	0.26	0.26	0.24	0.27
U.S. Route 219 Treatment MM 6.32	0.35	0.34	0.33	0.37	0.33	0.35	0.34	0.37
U.S. Route 219 Treatment MM 5.81	0.35	0.37	0.37	0.38	0.37	0.32	0.41	0.37
U.S. Route 219 Comparison	N/A	N/A	0.29	0.30	0.27	0.28	0.31	0.30

MM = Mile Marker
PC = Point of Curve
PT = Point of Tangent
N/A = Not Applicable

Table 36. SN65 Estimated available side friction first after treatment application.

Site	300 ft	200 ft	100 ft	PC	1/4	Midcurve	3/4	PT
WV Route 32 Treatment	0.29	0.39	0.40	0.67	0.62	0.61	0.57	0.53
WV Route 32 Comparison	0.30	0.30	0.34	0.32	0.31	0.31	0.32	0.35
U.S. Route 33 Treatment	0.33	0.30	0.35	0.57	0.56	0.47	0.52	0.54
U.S. Route 33 Comparison	N/A	N/A	0.23	0.25	0.24	0.27	0.24	0.28
U.S. Route 219 Treatment MM 6.32	0.32	0.30	0.27	0.64	0.52	0.52	0.54	0.56
U.S. Route 219 Treatment MM 5.81	0.34	0.35	0.31	0.60	0.56	0.57	0.56	0.61
U.S. Route 219 Comparison	N/A	N/A	0.28	0.29	0.24	0.27	0.25	0.31

MM = Mile Marker
PC = Point of Curve
PT = Point of Tangent
N/A = Not Applicable

Table 37. SN65 estimated available side friction second after treatment application.

Site	300 ft	200 ft	100 ft	PC	1/4	Midcurve	3/4	PT
WV Route 32 Treatment	0.33	0.35	0.33	0.54	0.51	0.47	0.46	0.53
WV Route 32 Comparison	0.40	0.34	0.32	0.34	0.26	0.26	0.27	0.33
U.S. Route 33 Treatment	0.30	0.32	0.32	0.47	0.42	0.35	0.42	0.40
U.S. Route 33 Comparison	N/A	N/A	0.31	0.27	0.27	0.27	0.27	0.28
U.S. Route 219 Treatment MM 6.32	0.31	0.33	0.31	0.54	0.54	0.55	0.56	0.57
U.S. Route 219 Treatment MM 5.81	0.36	0.37	0.40	0.56	0.51	0.47	0.55	0.55
U.S. Route 219 Comparison	N/A	N/A	0.30	0.34	0.27	0.30	0.34	0.33

MM = Mile Marker
PC = Point of Curve
PT = Point of Tangent
N/A = Not Applicable

A representative friction supply curve was derived for each test segment. The curves were plotted to illustrate the difference in friction supply before and after application of the HFST. Figure 54 through figure 56 show the plots. The figures support the discussion related to table 35 through table 37. While the available side friction is inconsistent from before to after at some comparison locations, the change in available side friction at 40 mph is quite substantial at the treatment sites. In general, the application appears to be inconsistent throughout the treatment curves, as discussed above. The available side friction is always highest at the PC and in three cases, reaches a minimum at approximately the midpoint of the treatment curve. At the WV Route 32 site, the friction continues to decrease through the curve and reaches a minimum at the PT (in the direction of data collection). This same trend occurred in the second after period, with the exception that the side friction does not decrease at the PT and was the same as in the first after period. The side friction remained consistent throughout the curve of the U.S. Route 219 Treatment MM 6.32.

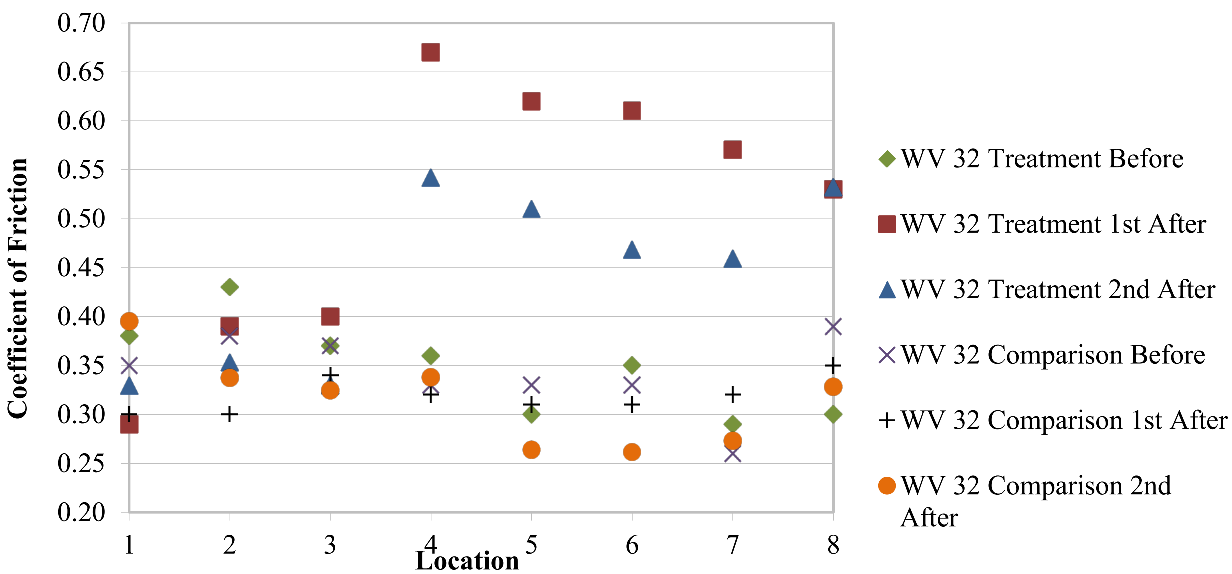


Figure 54. Graph. Side friction supply before and after for WV Route 32 sites.

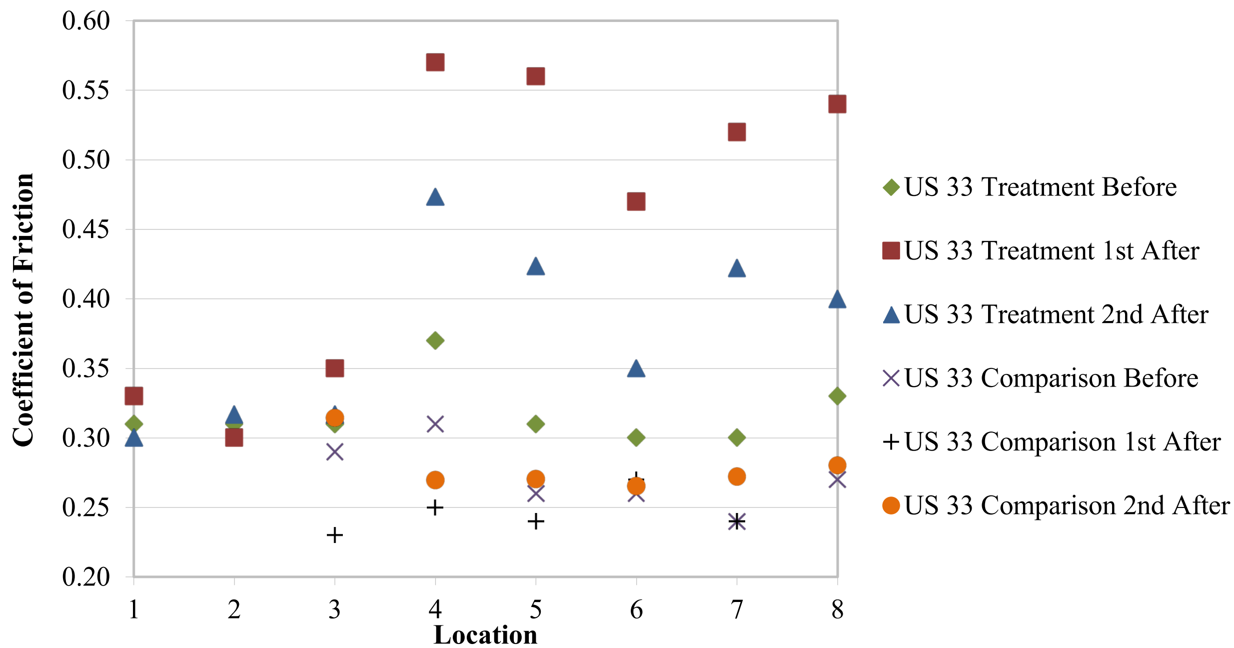


Figure 55. Graph. Side friction supply before and after for U.S. Route 33 sites.

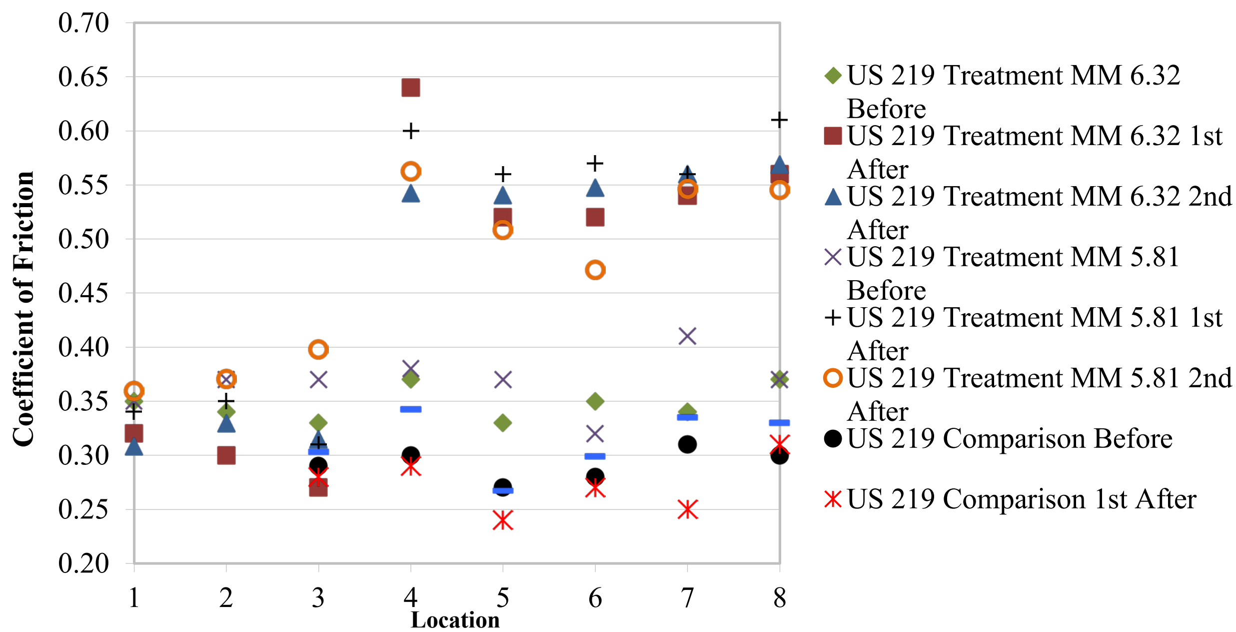


Figure 56. Graph. Side friction supply before and after for U.S. Route 219 sites.

In addition to the friction curves, the research team used speed data collected during the speed study to compute the difference in friction supply and the friction demanded by vehicles traversing horizontal curves at the treatment and control locations. The friction demand was computed using the point-mass model, using the equation shown in figure 57.

$$f = \frac{V^2}{15R} - \frac{e}{100}$$

Figure 57. Equation. Side friction demand.

Where:

f = side friction demand
V = vehicle operating speed (mph)
R = radius of horizontal curve (ft)
e = rate of superelevation (percent).

During the friction testing, the research team used a digital slope meter to measure the superelevation at the midpoint of each horizontal curve. The research team also compared descriptive statistics from the distributions of friction demand (e.g., mean, SD, 95th percentile) at each treatment and control curve location with the friction supply distribution. Table 38 shows the descriptive statistics for each horizontal curve observed.

Table 38. Descriptive statistics for passenger cars for margin of safety analysis.

Site	R	e	Passenger Car Speed				Side Friction Supply			
			Mean	SD	Min	Max	Mean	SD	Min	Max
WV Route 32 Treatment before	680	8	48.05	5.64	27	67	0.32	0.03	0.29	0.36
WV Route 32 Treatment 1st after	680	8	46.54	5.13	23	58	0.60	0.05	0.53	0.67
WV Route 32 Treatment 2nd after	680	8	47.63	7.21	17	69	0.50	0.04	0.46	0.54
WV Route 32 Comparison before	1,073	7	52.76	7.62	30	72	0.33	0.05	0.26	0.39
WV Route 32 Comparison 1st after	1,073	7	52.23	5.59	32	69	0.32	0.02	0.31	0.35
WV Route 32 Comparison 2nd after	1,073	7	52.48	6.06	24	75	0.32	0.05	0.26	0.40
U.S. Route 33 Treatment before	210	6	32.37	4.57	18	54	0.32	0.03	0.30	0.37
U.S. Route 33 Treatment 1st after	210	6	30.61	3.79	15	42	0.53	0.04	0.47	0.57
U.S. Route 33 Treatment 2nd after	210	6	30.15	5.02	15	54	0.41	0.04	0.35	0.47
U.S. Route 33 Comparison before	265	8.5	36.17	6.79	13	58	0.27	0.03	0.24	0.31
U.S. Route 33 Comparison 1st after	265	8.5	35.46	6.09	12	48	0.26	0.01	0.24	0.28
U.S. Route 33 Comparison 2nd after	265	8.5	34.44	5.45	11	48	0.28	0.02	0.27	0.31
U.S. Route 219 Treatment MM 6.32 before	605	11	36.29	7.22	12	62	0.35	0.03	0.33	0.37
U.S. Route 219 Treatment MM 6.32 1st after	605	11	40.34	5.43	13	56	0.55	0.05	0.52	0.64
U.S. Route 219 Treatment MM 6.32 2nd after	605	11	41.97	4.72	30	63	0.55	0.01	0.54	0.57
U.S. Route 219 Treatment MM 5.81 before	273	12	34.80	6.73	21	66	0.37	0.03	0.32	0.41
U.S. Route 219 Treatment MM 5.81 1st after	273	12	37.14	6.46	21	70	0.58	0.03	0.56	0.61
U.S. Route 219 Treatment MM 5.81 2nd after	273	12	33.72	5.20	21	47	0.53	0.04	0.47	0.56
U.S. Route 219 Comparison before	545	12	41.16	4.35	29	53	0.29	0.01	0.27	0.31
U.S. Route 219 Comparison 1st after	545	12	43.86	5.35	26	54	0.27	0.03	0.24	0.31
U.S. Route 219 Comparison 2nd after	545	12	41.12	5.80	27	48	0.31	0.03	0.27	0.34

MM = Mile Marker
SD = Standard Deviation
Max = Maximum
Min = Minimum

The margin of safety, based on the difference between the distributions of friction supply and demand, was computed for each treatment and control location, as table 39 shows. The mean friction demand at all sites, for all time periods, is lower than the mean side friction supply. The 85th percentile friction demand at all treatment sites, for all time periods, except the before time period of the U.S. 33 treatment site, is lower than the mean side friction supply. The 85th percentile friction demand for all time periods of the U.S. 33 comparison site is greater than the side friction supply. The 95th percentile friction demand at all treatment sites, for all time periods except the before time period of the U.S. Route 33 treatment site, is lower than the mean side friction supply. The 95th percentile friction demand for all time periods of the U.S. Route 33 comparison site is greater than the side friction supply. The only additional site where the friction demand at the 95th percentile exceeded the friction supply was at the U.S. Route 219 treatment MM 5.81 site in the before period.

Table 39. Friction demand and supply comparison.

	Side Friction Demand			Side Friction Supply			
Site	**Mean**	**85th Percentile**	**95th Percentile**	**Mean**	**SD**	**Min**	**Max**
WV Route 32 Treatment before	0.146	0.205	0.242	0.321	0.034	0.290	0.360
WV Route 32 Treatment 1st after	0.132	0.183	0.216	0.598	0.053	0.530	0.670
WV Route 32 Treatment 2nd after	0.142	0.218	0.267	0.502	0.037	0.459	0.542
WV Route 32 Comparison before	0.103	0.159	0.195	0.329	0.045	0.260	0.390
WV Route 32 Comparison 1st after	0.099	0.139	0.164	0.322	0.022	0.310	0.350
WV Route 32 Comparison 2nd after	0.101	0.144	0.172	0.315	0.046	0.261	0.395
U.S. Route 33 Treatment before	0.273	**0.377**	***0.445***	0.321	0.033	0.300	0.370
U.S. Route 33 Treatment 1st after	0.238	0.319	0.371	0.533	0.040	0.470	0.570
U.S. Route 33 Treatment 2nd after	0.229	0.337	0.409	0.414	0.045	0.350	0.473
U.S. Route 33 Comparison before	0.244	**0.385**	***0.479***	0.268	0.026	0.240	0.310
U.S. Route 33 Comparison 1st after	0.232	**0.354**	***0.435***	0.257	0.014	0.240	0.280
U.S. Route 33 Comparison 2nd after	0.214	**0.320**	***0.389***	0.279	0.018	0.265	0.314
U.S. Route 219 Treatment MM 6.32 before	0.035	0.101	0.146	0.352	0.027	0.330	0.370
U.S. Route 219 Treatment MM 6.32 1st after	0.069	0.123	0.158	0.554	0.049	0.520	0.640
U.S. Route 219 Treatment MM 6.32 2nd after	0.084	0.132	0.162	0.552	0.012	0.540	0.569
U.S. Route 219 Treatment MM 5.81 before	0.176	0.306	***0.394***	0.372	0.028	0.320	0.410
U.S. Route 219 Treatment MM 5.81 1st after	0.217	0.349	0.438	0.583	0.026	0.560	0.610
U.S. Route 219 Treatment MM 5.81 2nd after	0.158	0.254	0.317	0.527	0.037	0.472	0.563
U.S. Route 219 Comparison before	0.087	0.135	0.166	0.288	0.013	0.270	0.310
U.S. Route 219 Comparison 1st after	0.115	0.179	0.220	0.272	0.030	0.240	0.310
U.S. Route 219 Comparison 2nd after	0.087	0.152	0.194	0.313	0.028	0.267	0.342

Bold indicates 85th percentile friction demand exceeded mean side friction supply.
Italics indicates 95th percentile friction demand exceeded mean side friction supply.
SD = Standard Deviation
Max = Maximum
Min = Minimum

SUMMARY OF HFST EVALUATION FINDINGS

The research team performed operational, driver behavior, and friction evaluations on four treatment and three corresponding comparison sites in West Virginia. The operational and driver behavior analyses generally found no consistent differences at the treatment sites between the before and after time periods from the data. The friction analysis, however, clearly demonstrated that the friction increased considerably at the four horizontal curve treatment locations in West Virginia. The friction generally remained high for 1 year after the treatment was applied. A safety analysis being completed under a separate FHWA contract will reveal further information about the safety effects of the high-friction surface treatment.

5. SPEED EFFECTS OF OPTICAL SPEED BARS ON RURAL AND SUBURBAN ROADS

This section describes the OSB site selection criteria, design, site characteristics, and data collection and analysis methods used for evaluation.

OVERVIEW

OSBs are 18-inch-long and 12-inch-wide white transverse markings placed on both sides of the lane perpendicular to the centerline, edge line, or lane line in a pattern of progressively reduced spacing. The pattern gives drivers the impression that their speed is increasing. The intended outcome of the treatment is to reduce vehicle-operating speeds.

The initial and final spacing between the OSBs depends on the initial speed (approach speed) and desired speed in the curve (advisory speed). The length of roadway treated with the OSBs depends on the speed difference between the initial and final speed. FHWA recommends that drivers be in the OSB segment for at least 4 s.

OSBs can be painted on the roadway or a thermoplastic material is used to improve durability and increase longevity of the markings. OSBs are typically installed in school zones, at horizontal curve locations, or on tangent roadway segments. This project analyzed the effects of OSBs on vehicle operating speeds on horizontal curves using an observational before-after study. Figure 58 shows an example of an OSB treatment.

Source: KLS Engineering, LLC

Figure 58. Photo. OSB example.

SITE SELECTION

The research team used extensive outreach with several State and local transportation agencies to identify potential locations for installing OSB treatment on two-lane rural/suburban horizontal curves. The research team contacted more than 40 agencies to determine their willingness to install the treatment, 3 of which came forward to install or assist in installing the treatment for the study purposes. The three agencies are Massachusetts Southeastern Regional Planning and Economic Development District (SRPEDD), Mohave County (Arizona), and the Alabama Department of Transportation (ALDOT).

SRPEDD conducted road safety audits (RSA) in 2009 on eight roadways in suburban/rural areas of southeastern Massachusetts. The roadways were selected based on the number of lane-departure crashes resulting in injury or death. OSBs were one of the treatments recommended in the RSA, especially on the approaches to the many dangerous curves. Initially, the research team selected 11 curved segments on 6 different two-lane roadways for OSB consideration. Based on further discussions with SRPEDD and the counties, the initial list of 11 was narrowed to 8 locations on 4 roadways. During site visit, one location was found not to have an edge line and was thus eliminated, bringing the total number of sites in Massachusetts to seven. Table 40 shows the Massachusetts OSB treatment site(s) characteristics and location details.

Mohave County previously experimented with OSB at one location (Stockton Hill Road between MM 21 and 20) in July 2007, and was familiar with the OSB considerations. Mohave County examined speed and crash data maintained for its regional highway network and recommended four two-lane rural roadways as candidates. The locations had been the sites of a significant number of crashes, 85th percentile speeds significantly above posted, or both. Table 40 shows the Arizona OSB treatment site(s) characteristics and location details.

ALDOT provided 10 two-way rural roadways for OSB consideration. The roadways were selected based on crash data, at least three crashes on average per year over the last 10 years, on the approach, within, or immediately following a horizontal curve. Five roadways were deemed inappropriate for study purposes (residential streets, vicinity of major intersection, etc.). Eight curve segments on five different roadways were selected for OSB installation. Table 40 shows the Alabama OSB treatment site(s) characteristics. The identifying location information (e.g., route, milepost, county name) is not listed at the counties' request.

To summarize, a total of 19 treatment sites (7 in Massachusetts, 4 in Arizona, and 8 in Alabama) were selected for field evaluations.

TREATMENT DESIGN

The research team used two different types of OSB designs, as discussed in the following sections.

Design for Massachusetts and Arizona Sites

This layout was based on an experimental design by Mohave County Public Works (MCPW), which yielded positive results. The MCPW optical speed zone contained three speed-bar patterns—downstream, transition, and upstream—designed to convey to road users a sensory perception of increased speed while traveling through the zone. The MCPW design used a driver perception time of .75 s. The MCPW design layout involved computing the downstream bar-pattern spacing by multiplying the site-measured 85th percentile speed by 1 s and upstream bar spacing by 1.5 s (targeting a 50-percent increase in spacing from downstream to upstream). The transition pattern provided an incremental reduction in bar spacing, from upstream to downstream pattern spacing distance, between successive bars to disguise the physical change in bar spacing from the passing road user. The design used a fixed set of five bars in the upstream and downstream patterns while the transition set contained four bars.

To normalize the effects of differing PSLs, the research team slightly modified the MCPW design as follows:

- Used four bars each in the downstream pattern and upstream pattern and three bars in the transition zone.
- The upstream set bar spacing was established at 1-s headway.
- The downstream set bar spacing was established at 0.6-s headway.
- The transition set consisted of four sequential bars spaced at 0.9-, 0.8-, 0.7-, and 0.6-s headway.
- All spacing calculations were rounded to the next 5 ft.

This approach is demonstrated below using an example 40 mph, 85th percentile speed:

- Upstream set spacing at 1-s headway = (40*5280/3600)*1 ≈ 60 ft.
- Total upstream set spacing for four bars = 4*60 = 240 ft.
- Transition Set.
    - First bar spaced at 0.9-s headway ≈ 54 ft.
    - Second bar spaced at 0.8-s headway ≈ 48 ft.
    - Third bar spaced at 0.7-s headway ≈ 42 ft.
- Total transition set spacing = 54 ft +48 ft +42 ft = 144 ft.
- Total downstream set spacing at 0.6-s headway = 4*36 = 144 ft.
- Total treatment length = 240 ft + 144 ft + 144 ft = 528 ft.

Design for Alabama Sites

The design principle adopted for the Alabama sites was the same design used in studies by Katz and Arnold et al.[56,75] This design methodology considered an initial speed and a desired ending speed at each location. Based on these speeds, the length of OSB treatment is determined based on deceleration from the initial to the ending speed, and the bars are spaced such that a driver decelerating at a constant rate from the initial speed to the ending speed crosses four bars per second. The equation shown in figure 59 is used to determine the required length of the OSB treatment, and the equation shown in figure 60, developed by Katz, is used to find the spacing of the optical speed bar throughout the treatment.(76) A frequency of four bars per second was adopted for OSB design.

$$D = \frac{(v_0{}^2 - v_1{}^2)}{2a}$$

Figure 59. Equation. Length of OSB treatment.

Where:

D = distance traveled in slowing from v_0 to v_1.
a = deceleration rate.
v_0 = initial speed at the beginning of the treatment.
v_1 = final speed.

$$x = \frac{1}{2}a\left({}^{n}/_{f}\right)^2 + v_0\left({}^{n}/_{f}\right) + x_0$$

Figure 60. Equation. Individual placement of the OSBs.

Where:

x = placement of the optical speed bars.

x_0 = initial placement of the first bar. The value of x_0 is set to zero when a first bar is placed at the beginning of the treatment.

n = number of the optical speed bar for which the placement is determined.

f = required frequency of the bars, which is the number of OSBs seen in a second by motorists travelling through the treatment.

The dimensions of the OSB installed were in accordance with the 2009 *Manual on Uniform Traffic Control Devices* (MUTCD) (Part 3B.22) recommended guidelines. The markings were 12 inches wide by 18 inches long, installed on both sides of the lane perpendicular to the center line and edge line.

SITE CHARACTERISTICS

The range of features of the 19 study sites was quite broad. The characteristics of the study sites included the following:

- A minimum radius of 236 ft and maximum radius of 1,865 ft.
- Curves to the left and to the right.
- Speed limits of 25 to 55 mph.
- Advisory speeds of 20 to 35 mph.

Table 40 summarizes the site characteristics of the 19 treatment sites. The Before Period Data Summary section describes specific characteristics of each treatment site.

Table 40. OSB treatment location site characteristics.

	City/County	Route Name/ Number	Curve Direction/ Grade	AADT	PSL (mph)	Lane Widt h (ft)	L_c (ft)	Approx. Radius (ft)
1	Dolan Springs/ Mohave County, AZ	Pierce Ferry Road (northbound)	Left/Uphill	750	55/50[a]	12	993	497
2	Golden Valley/ Mohave County, AZ	Shinarump Road (southbound)	Left/Level	450	45	12	887	1,865
3	Meadview/Mohave County, AZ	Diamond Bar Road (southbound)	Right/Level	470	45	12	875	1,067
4	Golden Shores/ Mohave County, AZ	County Route 1 (southbound)	Right/Uphill	850	35	13	332	1,095
5	Alabama location #1	N/A	Right/Level	1,050	55	10	666	673
6	Alabama location #2	N/A	Right/Level	1,065	55	11	303	273
7	Alabama location #3	N/A	Right/Level	2,770	55	12	546	635
8	Alabama location #4	N/A	Right/Uphill	1,570	35	10	278	236
9	Alabama location #5	N/A	Right/ Downhill	380	40	10	1060	710
10	Alabama location #6	N/A	Right/Level	390	40	10	885	710
11	Alabama location #7	N/A	Right/Level	1,275	35	10	457	602
12	Alabama location #8	N/A	Right/ Downhill	1,370	35	11	223	486
13	Dartmouth/Bristol County, MA	Tucker Road (southbound)	Right/Level	2,900	30	13	817	1,099
14	Dartmouth/Bristol County, MA	Tucker Road (northbound)	Left/Level	3,950	35	12	293	569
15	Dartmouth/Bristol County, MA	Reed Road (southbound)	Left/Level	5,450	25	12	459	683
16	Fairhaven/Bristol County, MA	New Boston Road (southbound)	Left/Level	750	35	11	226	596
17	Fairhaven/Bristol County, MA	New Boston Road (northbound)	Right/Level	750	35	11	186	471
18	Rochester /Plymouth County, MA	Braley Hill Road (southbound)	Right/Level	1,150	30	11	673	695
19	Rochester/Plymouth County, MA	Braley Hill Road (northbound)	Right/Level	1,050	40	10	662	2,097

[a]-50 mph is nighttime PSL
AADT = Annual Average Daily Traffic
PSL = Posted Speed Limit
L_c = Length of Curve
Approx. = Approximate

INSTALLATION

In all three States, thermoplastic tape, applied with heat, was used to install the OSBs. Glass beads were added while the tape was being placed to increase visibility. Spacing of the bars was measured prior to their installation.

Installation at the four sites in Mohave County, AZ, occurred between October 25 and 30, 2012. Installation at the seven Massachusetts sites occurred between November 19 and 23, 2012. Each county installed the OSBs in their areas. The eight Alabama locations were installed between September 25 and October 1, 2013, and contractors performed the installation for ALDOT.

OPERATING SPEED EVALUATION METHODOLOGY

Data-Collection Procedures

The OSB treatments started on the tangent approaching the horizontal curves and ended near the beginning of the curve (PC station). An observational before-after study was employed to evaluate the OSB treatment. Speed data were collected before and after OSBs were applied to four sites in Arizona, eight sites in Alabama, and seven sites in Massachusetts. The before period data were collected prior to applying the OSBs. Two after-period data collection efforts were undertaken. The first after-period data were collected approximately 1 month after the OSBs were installed in Alabama, Arizona, and Massachusetts. The second after-period data were collected approximately 6 months after the first after period in Arizona and 3 months after the first after period in Alabama. There was no second after-period data collected at the Massachusetts sites. The second after period was performed to assess long-term novelty effects of the OSBs and to determine whether they became less effective in reducing vehicle operating speeds over time.

The research team used a control point on the same roadway to determine whether vehicle operating speeds remained constant at a location that was not treated to ensure that another factor was not influencing vehicle speeds. The control point locations for the OSB evaluation were points located between 0.2 and 0.8 mi upstream of the treatment site location, on a tangent segment. The control point location was far enough upstream of the treatment so drivers were unable to see the treatment from that location.

Data were collected using on-pavement traffic sensors. A total of six sensors, four in the OSB travel direction and two in the opposing lane, were placed to allow vehicles to be tracked throughout the study site, thus enabling the determination of speed changes for individual vehicles. The first sensor in the travel direction was placed at the control point. The second sensor in the travel direction was placed at the first transverse marking that delineated the OSBs, which was located on the curve approach. The third sensor in the travel direction was placed at the last transverse marking that delineated the OSBs, which was near the PC. The fourth sensor was placed at the curve midpoint to determine whether any speed reduction was maintained throughout the curve. Two sensors were placed in the opposing lane to determine whether the presence of a vehicle travelling in the opposite direction influenced driver speed choice. Figure 61 shows the layout of the speed data-collection equipment in relation to the OSB treatment.

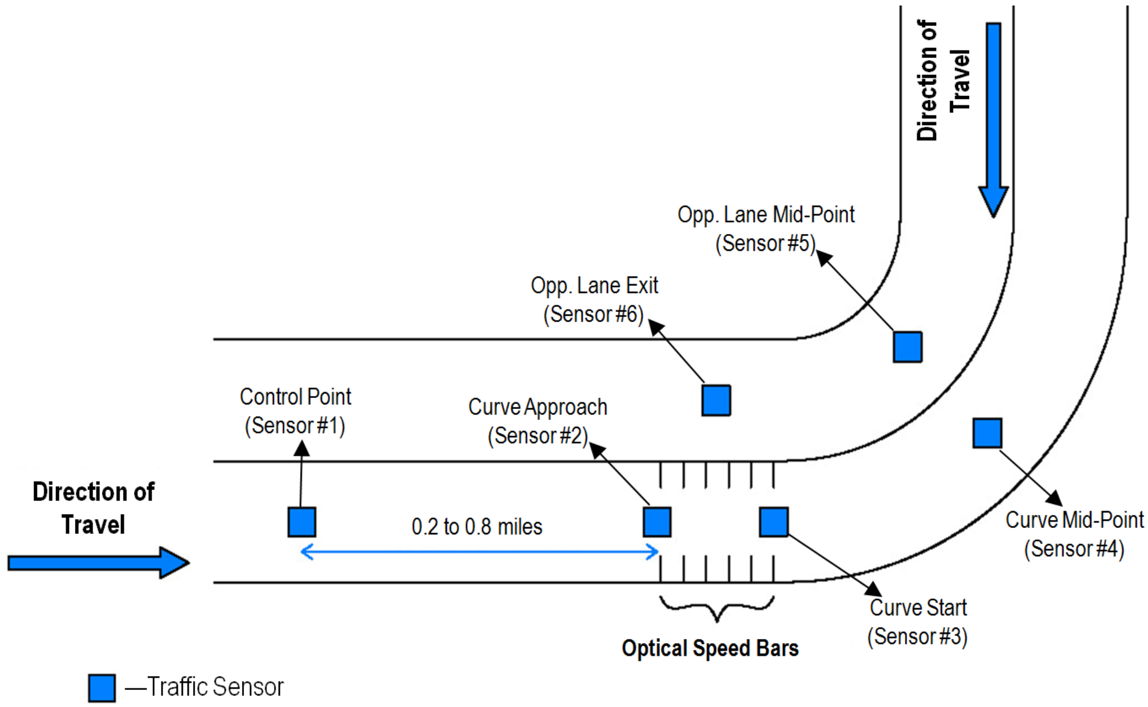


Figure 61. Diagram. OSB data-collection setup (not to scale).

Speed data were collected during both daytime and nighttime periods at each site. At each study site, the research team collected data during favorable driving conditions: clear weather with normal visibility (no fog) and dry roadway conditions, without the presence of standing water from an earlier rain. The data were screened to include only free-flow vehicles, defined as those vehicles traveling with a minimum headway of 5 s.(70,71) The analysis included only passenger vehicles, which were defined as vehicles having a length less than 24 ft.

The research team attempted to obtain the recommended 110 free-flow passenger car speeds for both the daytime and nighttime periods at each data-collection site in accordance with the sample size requirement described in table 7 and table 8. The 110 free-flow passenger car speeds recommended for each data-collection period based on the equation in figure 20 would yield 95-percent statistical significance and more than 90-percent statistical power, if a 2.0-mph difference in means was computed among the before and after samples. At sites where data were collected over 2 days/nights, and 1 day/night did not provide the recommended sample of free-flow passenger cars, a two-sample *t*-test was performed to compare the mean speeds between the 2 days/nights. If there was no statistically significant difference between the 2 days/nights, then they were combined to provide more free-flow speed observations for analysis.

An important data-collection condition to note relates to possible measurement error from the on-road sensors at several data-collection sites in the OSB evaluation. Outliers in the speed data were identified using the following process:

1. Speeds remained relatively consistent from the control point to approach at most sites, as would be expected, so it was assumed the speeds at these two points were precise.

2. At some sites, the speeds at the PC appeared to be unrealistically high, while at other sites, the speeds at the curve midpoint appeared to be unrealistically high. Many observations had acceleration rates exceeding 10 ft/s^2, which was nearly equivalent to deceleration rates assumed in stopping sight distance geometric design criteria. It was expected that vehicles either maintain their speed throughout the curve or decelerate, but not accelerate from the PC to curve midpoint. Previous research by Hu and Donnell found the maximum deceleration rate of vehicles entering a curve was 4.4 ft/s^2.[77] Using this previous study, any observation with an acceleration rate less than -4.5 ft/s or greater than 1.0 ft/s (to be conservative) from the approach to PC or from the approach to curve midpoint, was eliminated from the database.

3. While each vehicle speed was tracked from the control point location through the midpoint of the horizontal curve, in accordance with the data-collection setup shown in figure 61, the acceleration/deceleration rates between the approach point, PC location, and midcurve location were compared. When a point speed at the locations produced rates of acceleration/deceleration outside the limits identified in item 2 above, the research team excluded this individual point speed from the analysis.

Performance Measures and Analysis Methods

The performance measures used to assess the speed effects of the OSB treatment included the mean speed, difference in speeds between the beginning and end OSB speed measurement locations (delta *v* as described in the HFST Statistical Analysis section), speed variance, and proportion of vehicles exceeding the PSL. Details of the statistical analysis methods are the same as for HFST. The mean speed will be evaluated using the equation in figure 21. The proportion of vehicles exceeding the PSL will be assessed using the equations shown in figure 22 through figure 25, and the speed variance will be appraised using the equation shown in figure 26.

BEFORE PERIOD DATA SUMMARY

This section of the report describes the before period speeds at each data-collection location.

Arizona

This section describes the characteristics of the four treatment sites in Arizona.

Northbound Pierce Ferry Road, AZ

The research team collected data on Pierce Ferry Road in Meadview, AZ. The direction of travel for the data collection was northbound, and the curve direction was to the left. The radius of curve and the curve length, calculated using Google Earth™, were 497.40 ft and 992.78 ft, respectively. There was a substantial cut-slope on the inside of the curve, which limits horizontal sight distance along the curve. The daytime PSL was 55 mph, and nighttime PSL was 50 mph. As figure 62 shows, there was a curve warning sign (W1-2) with an advisory speed plaque of 35 mph (W13-1P) located prior to the PC. The travel lanes were 12 ft wide, and there were no paved shoulders on either side of the road. However, there was a gravel shoulder on both sides of the road approximately 5 ft wide.

The team collected speed data in the before period at the control point 0.4 mi before the curve approach, which was 800 ft before the PC. The research team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 660 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 62 shows the horizontal curve layout and the speed data-collection locations.

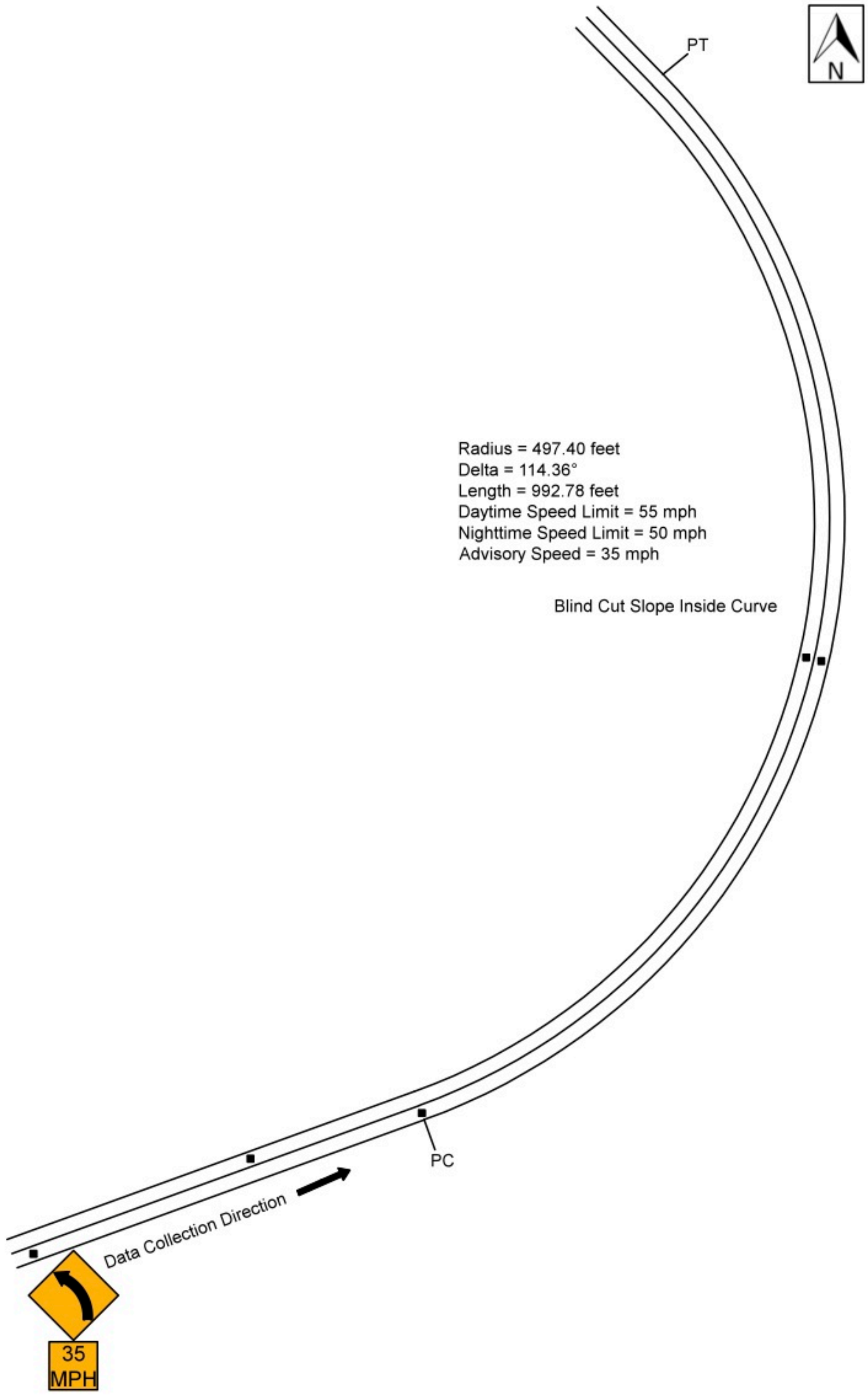


Figure 62. Diagram. Geometric layout of northbound Pierce Ferry Road (not to scale).

Table 41 shows the speed data collected in the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed (a vehicle present in the opposing lane) versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was a statistically significant difference in speeds at the control point, approach, PC, and curve midpoint for passenger cars versus heavy vehicles. There was also a statistically significant difference in speeds at the control point, approach, and PC for daytime passenger cars versus nighttime passenger cars. The nighttime speed was significantly lower at every data-collection point.

Table 41. Northbound Pierce Ferry Road before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	48.532	7.764	46.165	8.162	46.432	6.817	38.056	6.841	218	176	214
Passenger cars	**48.867**	7.565	**46.793**	7.855	**46.789**	6.628	**38.513**	6.739	203	166	199
Heavy trucks	**44.000**	9.220	**37.667**	7.697	**40.500**	7.546	**32.000**	5.237	15	10	15
Daytime passenger cars	**49.577**	7.116	**47.521**	7.429	**47.669**	6.277	**39.271**	6.744	142	118	140
Nighttime passenger cars	**47.213**	8.347	**45.098**	8.594	**44.625**	7.028	**36.712**	6.430	61	48	59
Opposed passenger cars	47.806	6.122	46.444	8.833	47.621	7.683	38.029	6.671	36	29	35
Unopposed passenger cars	49.096	7.838	46.868	7.655	46.613	6.400	38.616	6.769	167	137	164

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 63 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 63 also shows the mean speed for trucks within the study section. The figure shows that mean speeds for both heavy trucks and passenger cars remain relatively stable from the approach to the PC, but decrease substantially from the PC to the midpoint of the curve. The heavy-truck mean speeds align more with the 15th percentile passenger car speeds. Both the passenger car and truck mean speeds are consistent with the advisory speed of 35 mph at the midpoint of the curve. The mean acceleration rate from the PC to the midpoint of the curve was -3.534 ft/s for passenger cars and -3.803 ft/s for trucks. A negative value indicates deceleration.

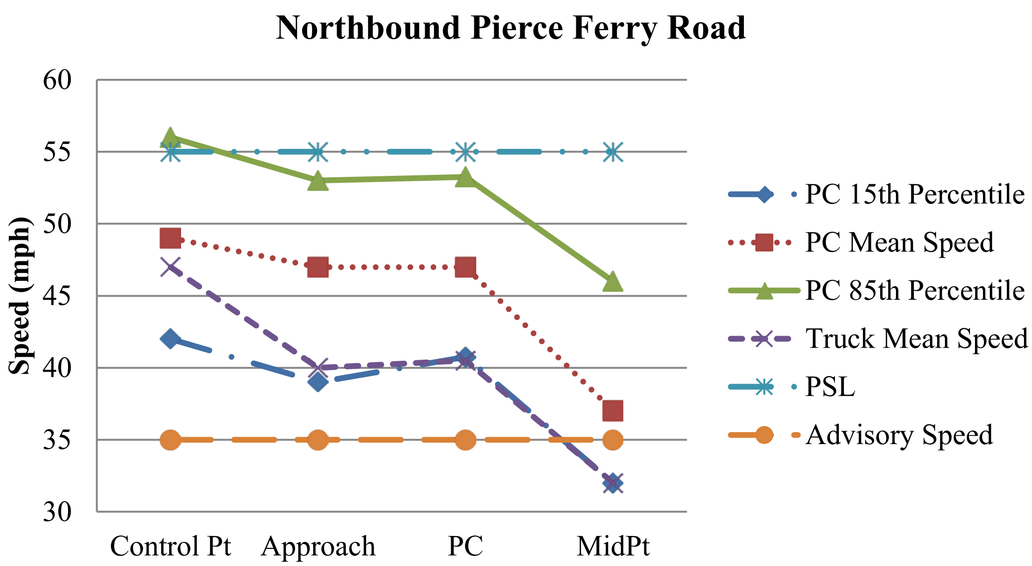


Figure 63. Graph. Graphical representation of speeds on northbound Pierce Ferry Road.

Southbound Shinarump Road, AZ

The research team collected data on Shinarump Road in Golden Valley, AZ. The direction of travel for the data collection was southbound, and the curve direction was to the left. The radius of curve and the curve length, calculated using Google Earth™, were 1,865.05 ft and 887.35 ft, respectively. The PSL was 45 mph. The travel lanes were 12 ft wide, and there was no shoulder on either side of the road.

The team collected speed data at the control point, 0.6 mi before the curve approach, which was 800 ft before the PC. The research team also collected speed data at the PC and midpoint of the curve. One sensor was placed in the opposing direction of travel at the midpoint of the curve, and the other was placed 618 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 64 shows the horizontal curve layout, along with the speed collection locations.

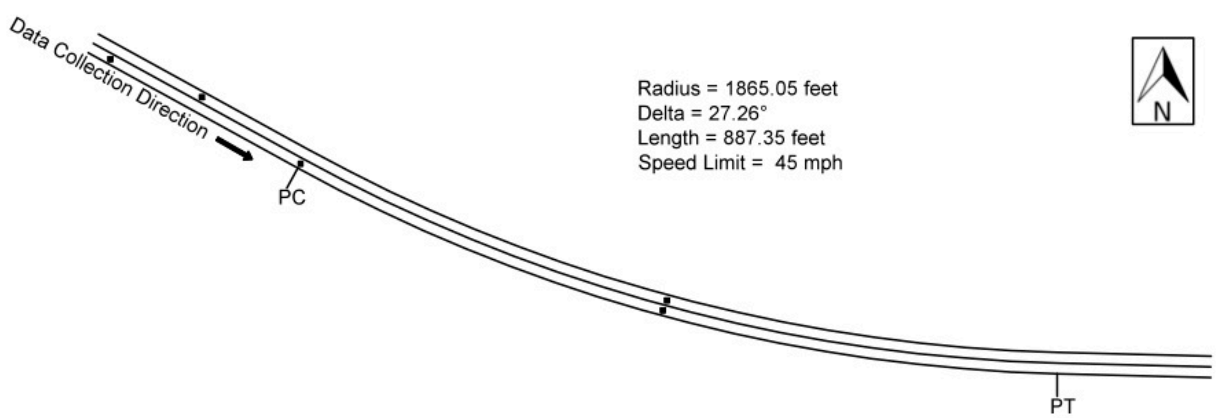


Figure 64. Diagram. Geometric layout of southbound Shinarump Road (not to scale).

Table 42 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each curve location. For this treatment site, there was a statistically significant difference in speeds at the PC location for passenger cars versus heavy vehicles. There was also a statistically significant difference at the control point and PC for daytime versus nighttime passenger cars. There was a statistically significant difference at the approach and PC locations for opposed versus unopposed passenger cars.

Table 42. Southbound Shinarump Road before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	47.317	6.928	51.099	8.603	47.299	6.864	51.100	8.042	303	288	209
Passenger cars	47.566	6.768	51.241	8.023	**48.017**	6.341	51.537	7.773	249	238	175
Heavy trucks	46.167	7.583	50.444	10.956	**43.880**	8.188	48.853	9.099	54	50	34
Daytime passenger cars	**46.686**	7.001	50.777	8.431	**47.009**	6.033	50.894	7.375	121	115	94
Nighttime passenger cars	**48.398**	6.458	51.680	7.624	**48.959**	6.500	52.284	8.193	128	123	81
Opposed passenger cars	45.967	6.646	**48.200**	5.580	**45.933**	4.982	50.875	6.556	30	30	24
Unopposed passenger cars	47.785	6.770	**51.658**	8.224	**48.317**	6.468	51.642	7.964	219	208	151

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 65 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 65 also shows the mean speed for trucks within the study section. As the figure shows, the patterns of the speed changes were similar for the passenger car speeds and the truck mean speeds on the approach to curve. The speeds for passenger cars and the truck mean speeds increased from the control point to the approach of the curve, decreased from the approach to the PC, and then increased again from the PC to the midpoint of the curve. The 85th percentile speeds and the mean speeds for passenger cars along the curve were higher than the PSL of 45 mph. The mean acceleration rate from the PC to the midpoint of the curve was 4 ft/s for passenger cars and 6 ft/s for trucks.

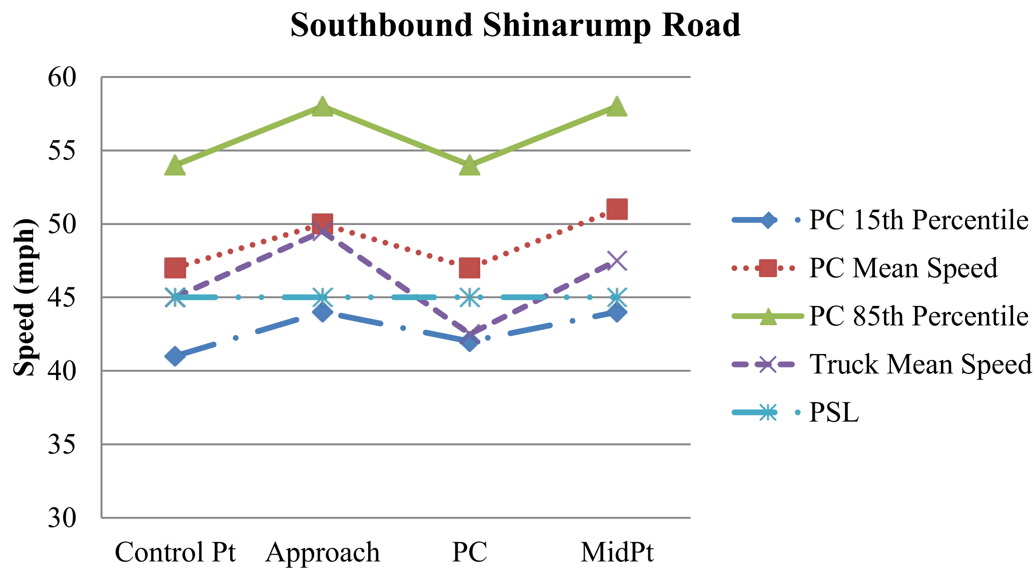


Figure 65. Graph. Graphical representation of speeds on southbound Shinarump Road.

Southbound Diamond Bar Road, AZ

The research team collected data on Diamond Bar Road in Meadview, AZ. The direction of travel for the data collection was in the southbound direction, and the curve direction was to the right. The radius of curve and the curve length, calculated using Google Earth™, were 1,067.25 ft 875.32 ft, respectively. There were desert plants on the inside of the curve, which limited horizontal sight distance along the curve. The PSL was 45 mph. As figure 66 shows, there was a curve warning sign (W1-2) located after the PC. The travel lanes were 12 ft wide, and there were 5-ft paved shoulders on both sides of the road.

The team collected speed data at the control point, 0.4 mi before the curve approach, which was 800 ft before the PC. Speed data were also collected at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 640 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 66 shows the horizontal curve layout, along with the speed collection locations.

The OSBs were installed differently at this site compared with the other sites. The OSB treatment started 400 ft upstream of the curve start, compared with beginning at the curve start at the other Arizona, Massachusetts, and Alabama sites. Because the treatment was offset from the curve PC location, an additional sensor was placed at the end of the OSB treatment. Figure 67 shows the layout of the sensors in the after periods.

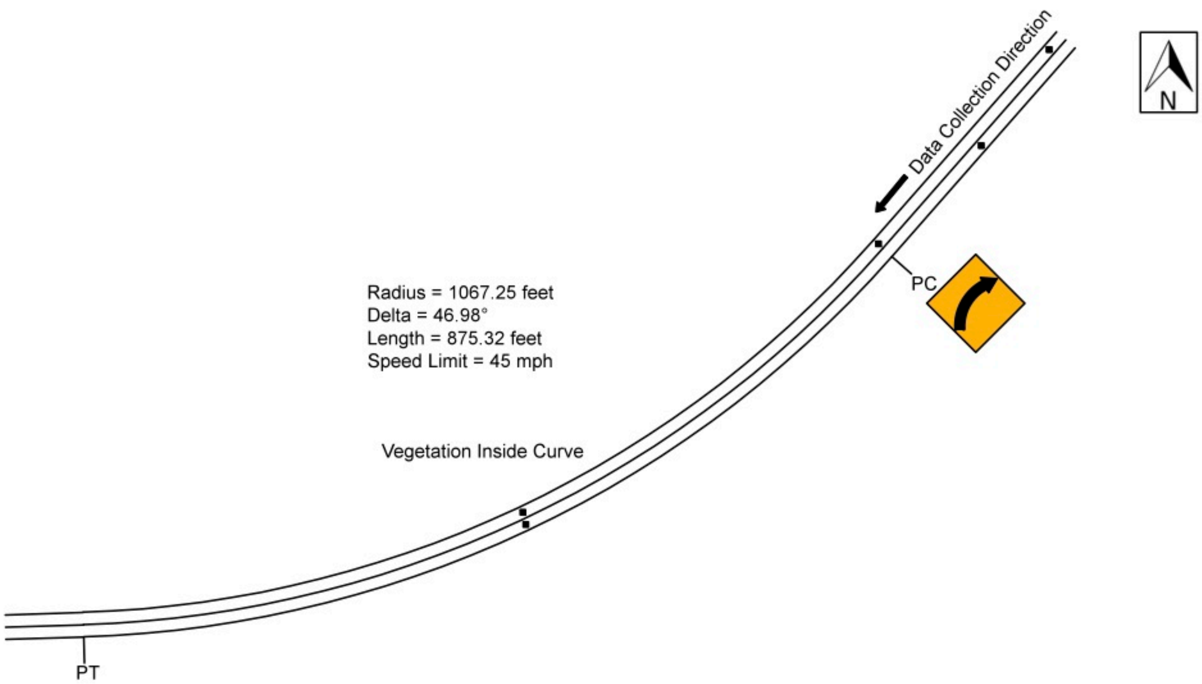


Figure 66. Diagram. Geometric layout of southbound Diamond Bar Road in the before period (not to scale).

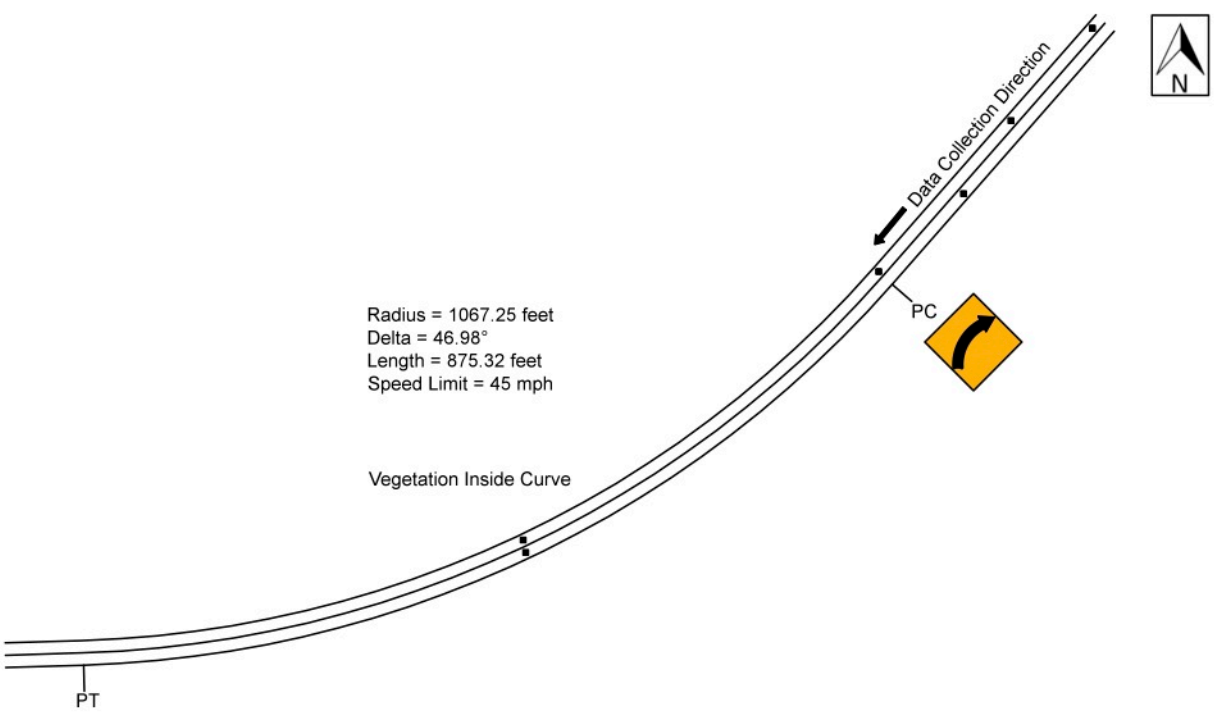


Figure 67. Diagram. Geometric layout of southbound Diamond Bar Road in the after periods (not to scale).

Table 43 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team again performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was a statistically significant difference in speeds at the control point, approach, PC, and curve midpoint for passenger cars versus heavy vehicles. The heavy vehicle speed was lower than the passenger car speed at all the data-collection points. There also was a significant difference at the PC for daytime versus nighttime passenger cars.

Table 43. Southbound Diamond Bar Road before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	53.421	8.312	49.171	6.730	50.299	7.116	50.435	7.621	292	214	209
Passenger cars	**53.867**	8.389	**49.516**	6.675	**50.874**	6.996	50.703	7.611	256	182	182
Heavy trucks	**50.250**	7.064	**46.722**	6.704	**47.031**	7.014	48.630	7.581	36	32	27
Daytime passenger cars	54.061	8.582	49.873	7.057	**51.706**	7.113	51.109	7.876	181	126	128
Nighttime passenger cars	53.400	7.939	48.653	5.598	**49.000**	6.399	49.741	6.918	75	56	54
Opposed passenger cars	53.472	9.376	50.057	6.320	51.000	8.218	50.057	8.359	53	35	35
Unopposed passenger cars	53.970	8.134	49.374	6.773	50.844	6.705	50.857	7.445	203	147	147

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 68 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 68 also shows the mean speed for trucks within the study section. As the figure shows, the passenger car speeds and the truck mean speeds were relatively stable along the curve. The speed changes between points were minor. The truck mean speeds were consistent from the control point to the PC; however, the speeds increased slightly from the PC to midpoint of the curve. The 85th percentile speeds and mean speeds for passenger cars are higher than the PSL of 45 mph. The mean acceleration rate from the PC to the midpoint of the curve was 1 ft/s for passenger cars and 2.835 ft/s for trucks.

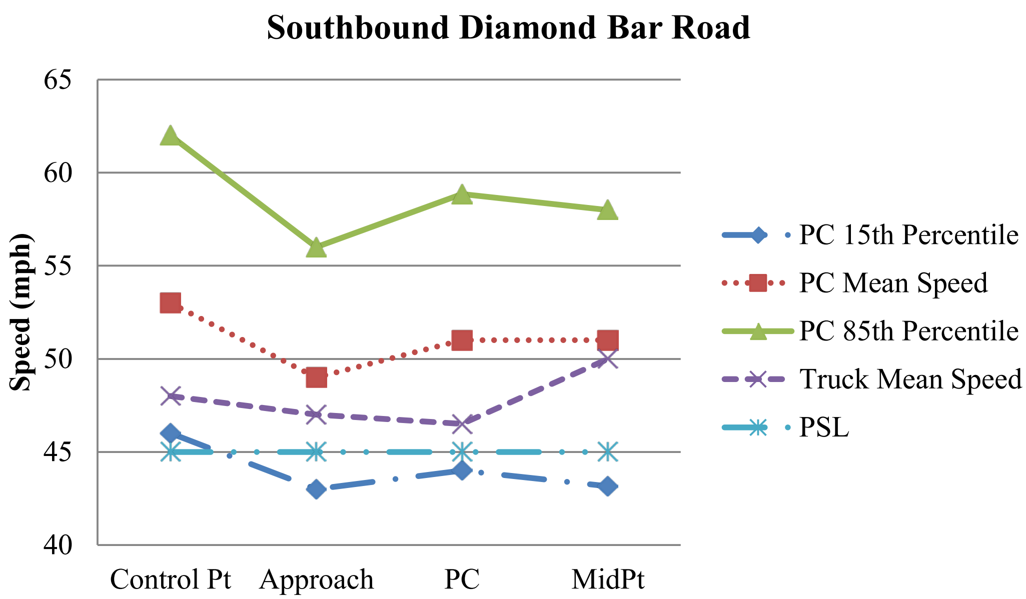


Figure 68. Graph. Graphical representation of speeds on southbound Diamond Bar Road.

Southbound County Route 1, AZ

The research team collected data on County Route 1 in Golden Shores, AZ. The direction of travel for the data collection was southbound, and the curve direction was to the right. The curve was located on a moderate upgrade. The radius of curve and the curve length, calculated using Google Earth, were 1,095.39 ft and 332.27 ft, respectively. The PSL was 35 mph. The travel lanes were 13 ft wide, and there was a 4- to 5-ft paved shoulder on both sides of the road.

The team collected speed data at the control point, 0.8 mi before the curve approach, which was 800 ft before the PC. The research team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 732 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 69 shows the horizontal curve layout, along with the speed collection locations.

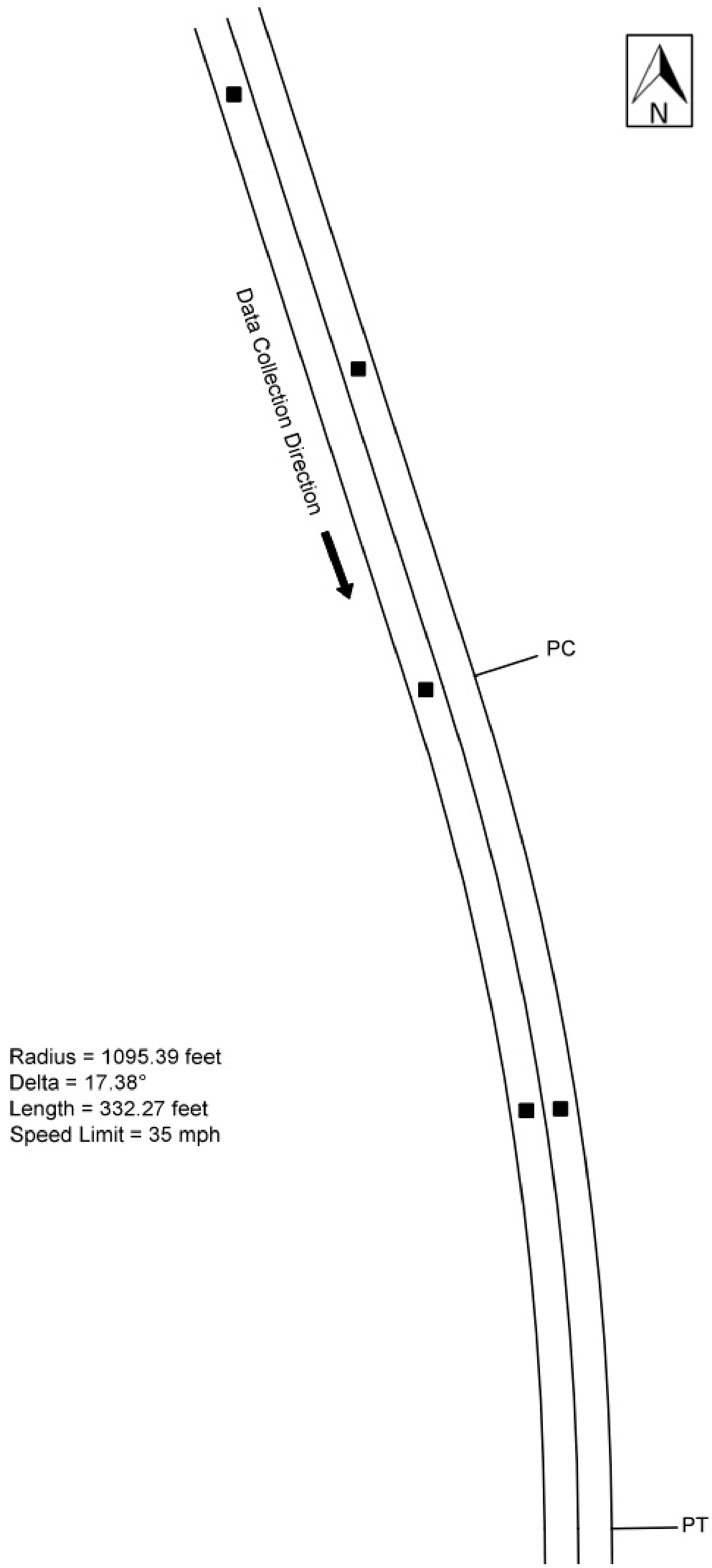


Figure 69. Diagram. Geometric layout of southbound County Route 1 (not to scale).

Table 44 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each curve location. For this treatment site, there was a statistically significant difference in speeds at the approach, PC, and midpoint for daytime versus nighttime passenger cars. There was also a significant difference at the midpoint for opposed versus unopposed passenger cars.

Table 44. Southbound County Route 1 before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	53.471	8.882	47.731	5.559	44.021	5.211	43.919	7.259	342	329	307
Passenger cars	53.532	8.901	47.810	5.515	44.134	5.125	43.918	7.262	327	314	292
Heavy trucks	52.133	8.651	46.000	6.414	41.667	6.532	43.933	7.450	15	15	15
Daytime passenger cars	53.847	9.266	**49.038**	5.430	**44.977**	4.895	**44.599**	7.081	183	174	162
Nighttime passenger cars	53.132	8.428	**46.250**	5.238	**43.086**	5.228	**43.069**	7.421	144	140	130
Opposed passenger cars	53.479	9.299	47.125	5.354	44.111	4.900	**41.091**	7.844	48	45	44
Unopposed passenger cars	53.541	8.848	47.928	5.543	44.138	5.171	**44.419**	7.053	279	269	248

Bold indicates significance at 95 percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 70 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 70 also shows the mean speed for trucks within the study section. The figure shows the passenger car speeds decelerated substantially from the control point to the PC and then stabilized from the PC to the midpoint of the curve. The truck mean speeds decreased from the control point to the midpoint of the curve. The passenger car speeds and the truck mean speeds along the curve were higher than the PSL of 35 mph. The mean acceleration rate from the PC to the midpoint of the curve was 0.581 ft/s for passenger cars and 0.668 ft/s for trucks.

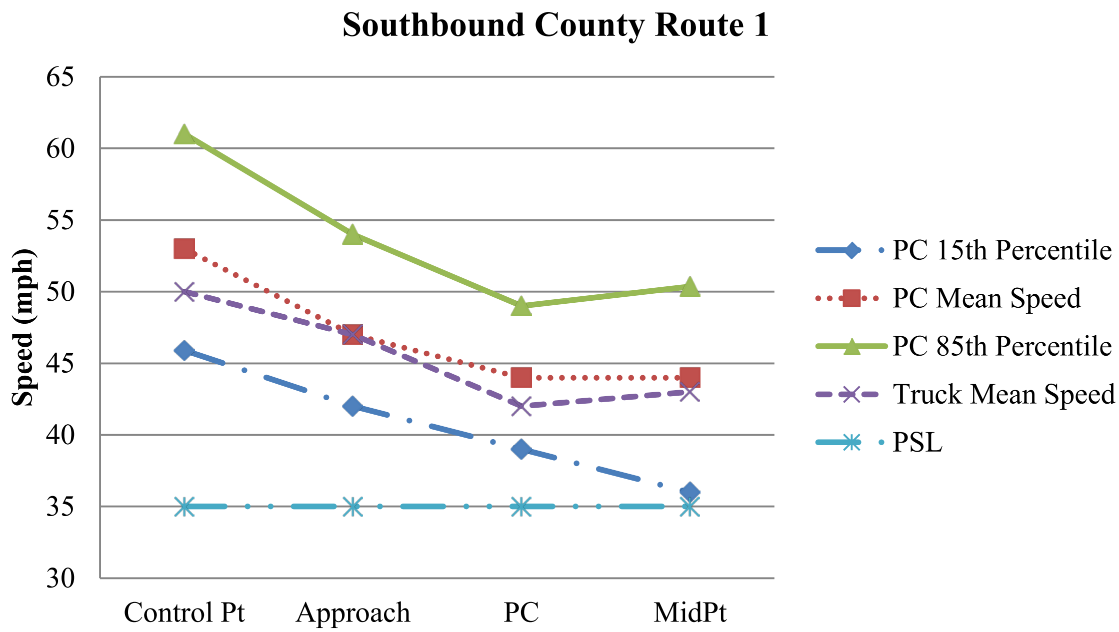


Figure 70. Graph. Graphical representation of speeds on southbound County Route 1.

Alabama

This section describes the characteristics of the eight treatment sites in Alabama. ALDOT had an agreement with the counties that route information would not be disclosed. Thus, in the discussion below all eight Alabama locations are referenced numerically (1 through 8).

Alabama Location #1

The direction of travel for the data collection was northbound, and the curve direction was to the right. The radius of curve and the curve length, calculated using Google Earth™, were 672.84 ft and 665.84 ft, respectively. The PSL was 55 mph. As figure 71 shows, there was a winding road sign (W1-5) 365 ft before the curve approach. The travel lanes were 11 ft wide, and there were 2-ft paved shoulders on both sides of the road. There were trees offset approximately 10 ft from the inside edge of pavement, which limited horizontal sight distance along the curve.

The research team collected speed data at the control point, 0.4 mi before the curve approach, which was 500 ft before the PC. The team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 515 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 71 shows the horizontal curve layout, along with the speed data-collection locations.

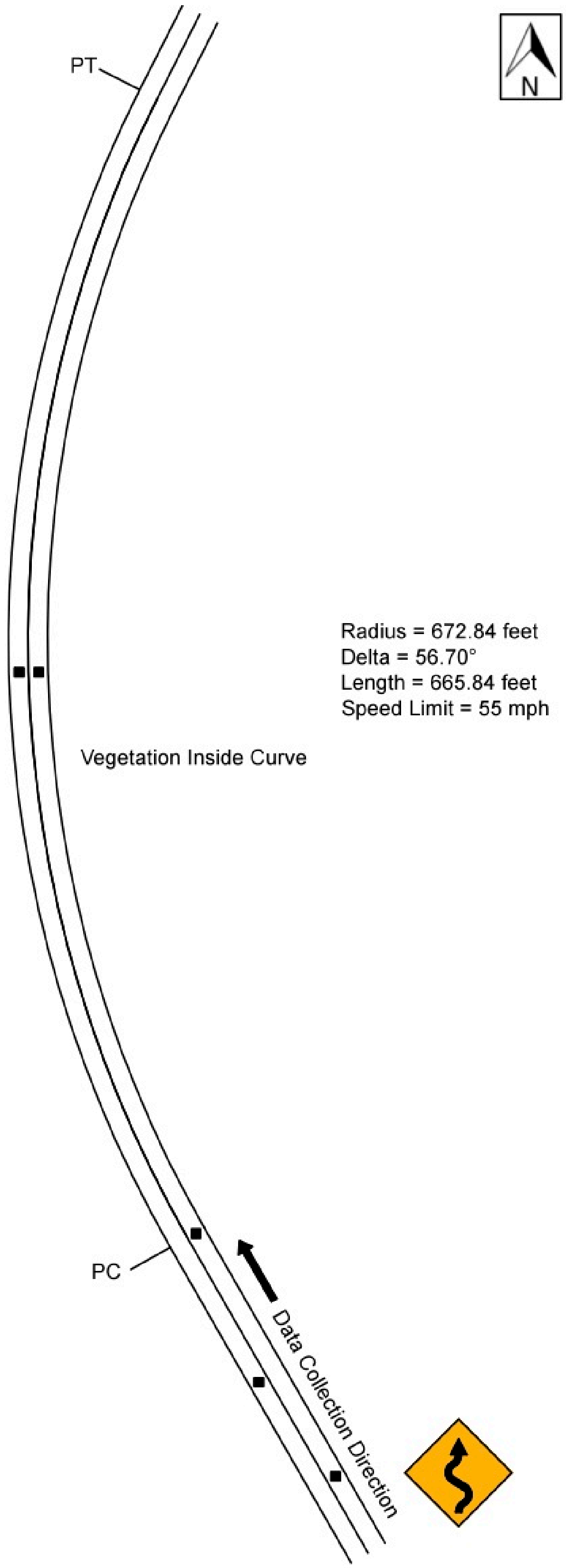


Figure 71. Diagram. Geometric layout of Alabama Location #1 (not to scale).

Table 45 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was a statistically significant difference in speeds at the midpoint for passenger cars versus heavy trucks. The only other statistically significant difference in speeds was at the PC for opposed versus unopposed passenger cars.

Table 45. Alabama Location #1 before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	47.883	9.368	49.208	7.596	50.136	8.725	42.323	6.912	375	88	353
Passenger cars	47.917	9.380	49.218	7.540	50.429	8.700	**42.516**	6.892	362	84	341
Heavy trucks	46.923	9.332	48.923	9.394	44.000	7.789	**36.833**	5.132	13	4	12
Daytime passenger cars	47.638	9.418	48.805	7.641	50.795	8.551	42.288	6.764	174	39	160
Nighttime passenger cars	48.176	9.362	49.601	7.444	50.111	8.912	42.718	7.016	188	45	181
Opposed passenger cars	47.529	10.661	48.793	7.385	**46.833**	7.648	42.037	6.512	87	24	82
Unopposed passenger cars	48.040	8.955	49.353	7.596	**51.867**	8.736	42.668	7.014	275	60	259

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 72 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 72 also shows the mean speed for trucks within the study section. The figure shows the speeds for passenger cars increased slightly from the control point to the PC and decreased substantially from the PC to the midpoint of the curve. The truck mean speeds were relatively consistent from the control point the PC and dropped substantially from the PC to the midpoint of the curve. The 85th percentile speeds for passenger cars at the control point, the approach, and the PC were higher than the PSL of 55 mph. Only at the midpoint of the curve were the 85th percentile speeds for passenger cars lower than the PSL of 55 mph. The mean speeds and 15th percentile speeds for passenger cars and the truck mean speeds along the curve were all lower than the PSL of 55 mph. The mean acceleration rate from the PC to the midpoint of the curve was -8.726 ft/s for passenger cars and -6.304 ft/s for trucks. A negative value indicates deceleration.

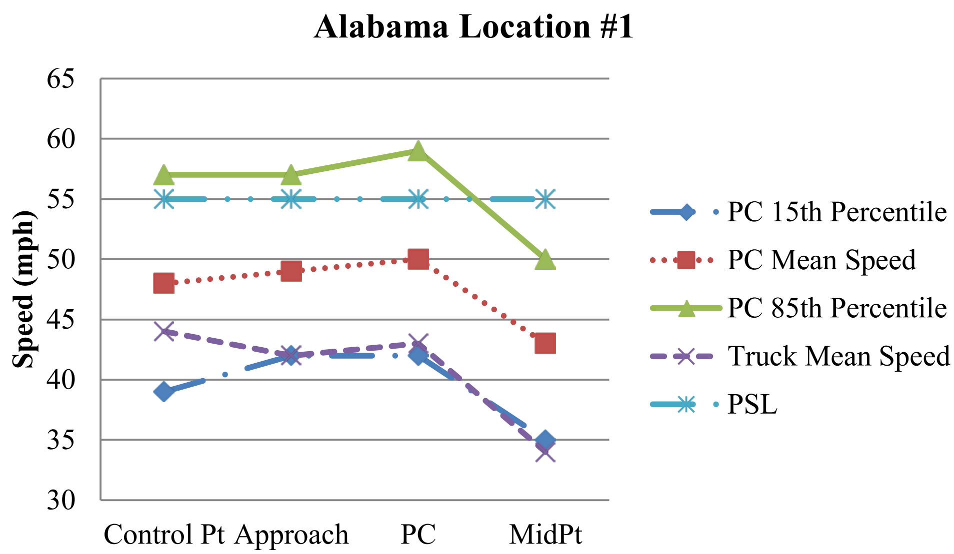


Figure 72. Graph. Graphical representation of speeds at Alabama Location #1.

Alabama Location #2

This curve was located approximately 0.4 mi upstream of the Alabama Location #1 curve. The direction of travel for the data collection was southbound, and the curve direction was to the right. The curve approach and curve were located on a downgrade. The radius of curve and the curve length, calculated using Google Earth™, were 272.92 ft and 303.38 ft. respectively. There was a substantial cut slope with trees on the inside of the curve, which limited horizontal sight distance along the curve. The PSL was 55 mph. As figure 73 shows, there was a winding road sign (W1-5) with an advisory speed plaque (W13-1P) of 30 mph 175 ft before the PC. The travel lanes were 11 ft wide, and there were 2-ft paved shoulders on both sides of the road. There was a drainage ditch located on the inside of the curve and guardrail on the outside of the curve with chevron markers.

The research team collected speed data at the control point, 800 ft before the curve approach, which was 500 ft before the PC. The team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 440 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 73 shows the horizontal curve layout, along with the speed data-collection locations.

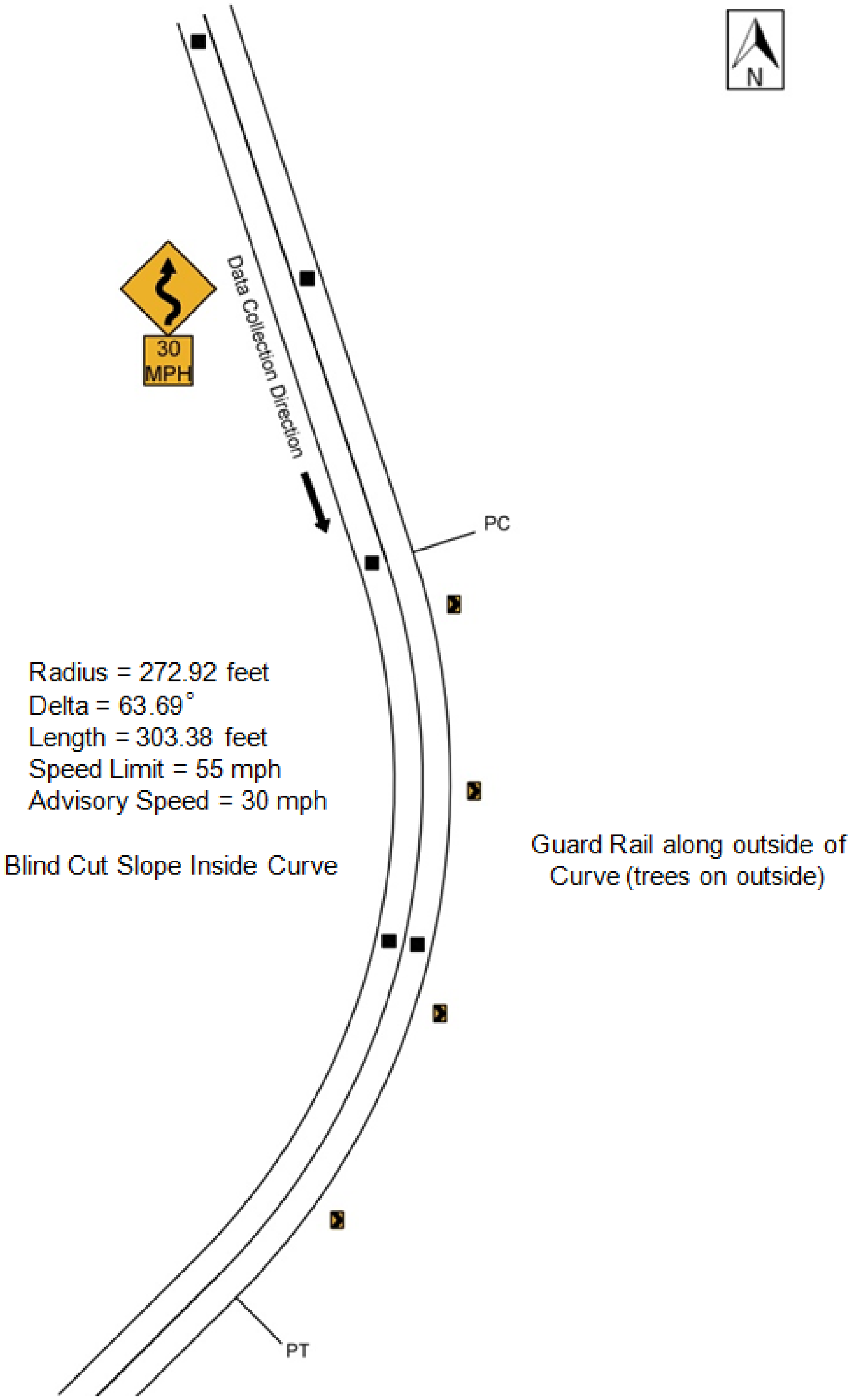


Figure 73. Diagram. Geometric layout of Alabama Location #2 (not to scale).

Table 46 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was a statistically significant difference in speeds at the control point and approach for passenger cars versus heavy vehicles. There was also a statistically significant difference in speeds at the control point, approach, and curve midpoint for unopposed passenger cars versus opposed passenger cars.

Table 46. Alabama Location #2 before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	49.269	7.604	44.291	5.822	42.472	5.747	38.337	7.101	398	290	359
Passenger cars	**49.569**	7.238	**44.479**	5.623	42.630	5.554	38.418	7.137	376	273	337
Heavy trucks	**44.136**	11.319	**41.091**	8.053	39.941	8.050	37.091	6.560	22	17	22
Daytime passenger cars	49.884	8.070	45.072	5.722	43.263	5.737	37.821	7.802	138	99	123
Nighttime passenger cars	49.387	6.721	44.134	5.548	42.270	5.432	38.762	6.720	238	174	214
Opposed passenger cars	**47.809**	7.949	**43.022**	6.312	41.721	5.057	**36.378**	7.241	89	61	82
Unopposed passenger cars	**50.115**	6.928	**44.930**	5.323	42.892	5.674	**39.075**	6.991	287	212	255

Bold indicates significance at 95 percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 74 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 74 also shows the mean speed for trucks within the study section. The figure shows the patterns of the speed changes were similar for the passenger car speeds and the truck mean speeds at this curve location. The speeds for passenger cars and the truck mean speeds decreased consistently from the control point to the midpoint of the curve. The speeds for passenger cars and the truck mean speeds along the curve were lower than the PSL of 55 mph but higher than the advisory speed of 30 mph. The mean acceleration rate from the PC to the midpoint of the curve was -2.128 ft/s for passenger cars and -2.211 ft/s for trucks. A negative value indicates deceleration.

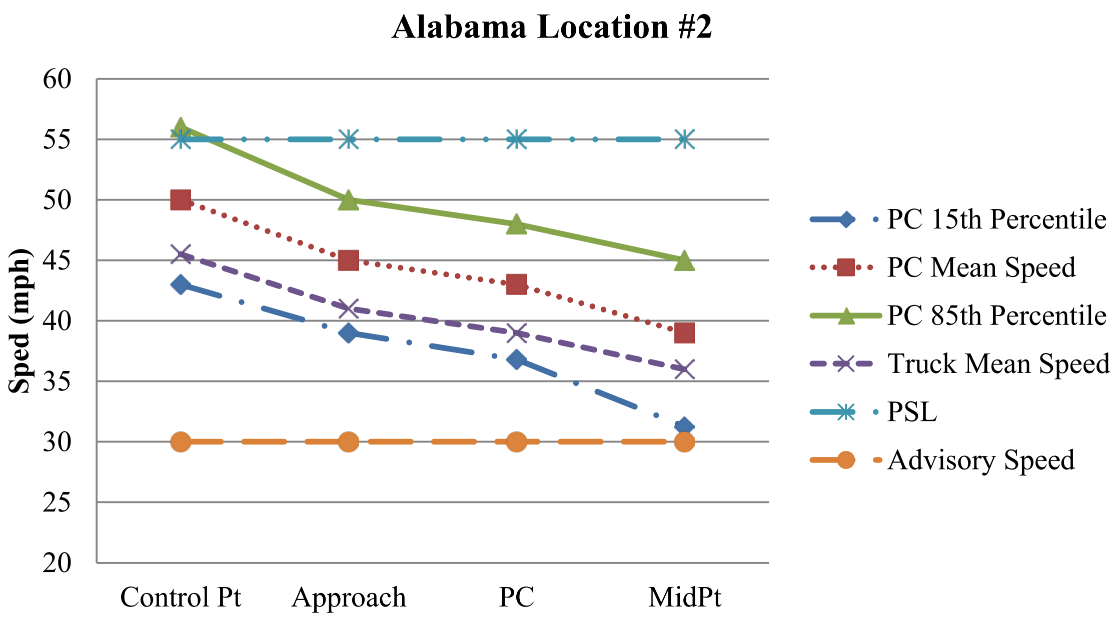


Figure 74. Graph. Graphical representation of speeds at Alabama Location #2.

Alabama Location #3

The direction of travel for the data collection was southbound, and the curve direction was to the right. This curve to the right was part of a reverse curve, with a curve back to the left immediately after. The radius of curve and the curve length, calculated using Google Earth™, were 635.15 ft and 545.62 ft, respectively. The PSL was 55 mph. As figure 75 shows, there was a winding road sign (W1-5) with an advisory speed plaque (W13-1P) of 35 mph positioned 460 ft before the PC; there were also chevrons on the outside of the curve. The travel lanes were 12 ft wide, and there were 2-ft paved shoulders on either side of the road.

The research team collected speed data at the control point, 0.2 mi before the curve approach, which was 500 ft before the PC. The team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 500 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 75 shows the horizontal curve layout, along with the speed data-collection locations.

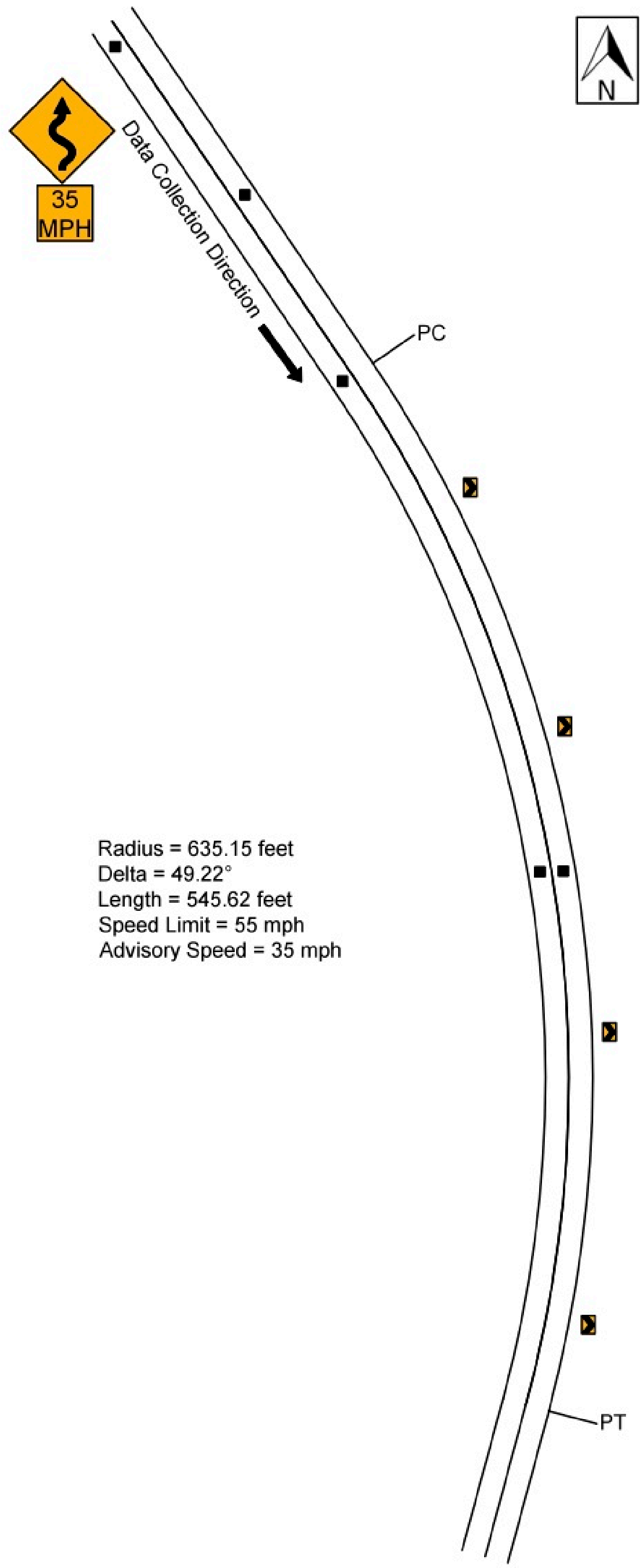


Figure 75. Diagram. Geometric layout of Alabama Location #3 (not to scale).

Table 47 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was a statistically significant difference in speeds at the approach, PC, and the curve midpoint for passenger cars versus heavy vehicles. There was also a statistically significant difference in speeds at the approach for daytime passenger cars versus nighttime passenger cars.

Table 47. Alabama Location #3 before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	65.122	13.222	49.842	6.461	41.329	4.665	41.192	5.936	499	474	474
Passenger cars	65.372	13.359	**50.011**	6.519	**41.652**	4.653	**41.691**	5.714	441	417	417
Heavy trucks	63.224	12.070	**48.552**	5.891	**38.965**	4.066	**37.544**	6.299	58	57	57
Daytime passenger cars	66.174	13.109	**50.566**	6.965	41.960	4.681	41.609	6.025	265	248	253
Nighttime passenger cars	64.165	13.675	**49.176**	5.702	41.201	4.589	41.817	5.214	176	169	164
Opposed passenger cars	65.516	13.031	49.979	6.684	41.449	5.061	41.928	6.036	188	176	180
Unopposed passenger cars	65.265	13.622	50.036	6.408	41.801	4.336	41.511	5.463	253	241	237

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 76 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 76 also shows the mean speed for trucks within the study section. As the figure shows, the passenger car speeds and truck mean speeds decelerated substantially from the control point to the PC. Both passenger car speeds and the truck mean speeds then stabilized from the PC to midpoint of the curve. The passenger car speeds and the truck mean speeds along the curve were all higher than the advisory speed of 35 mph. The mean acceleration rate from the PC to the midpoint of the curve was 0.321 ft/s for passenger cars. The truck acceleration rate remained the same from the PC to the midpoint.

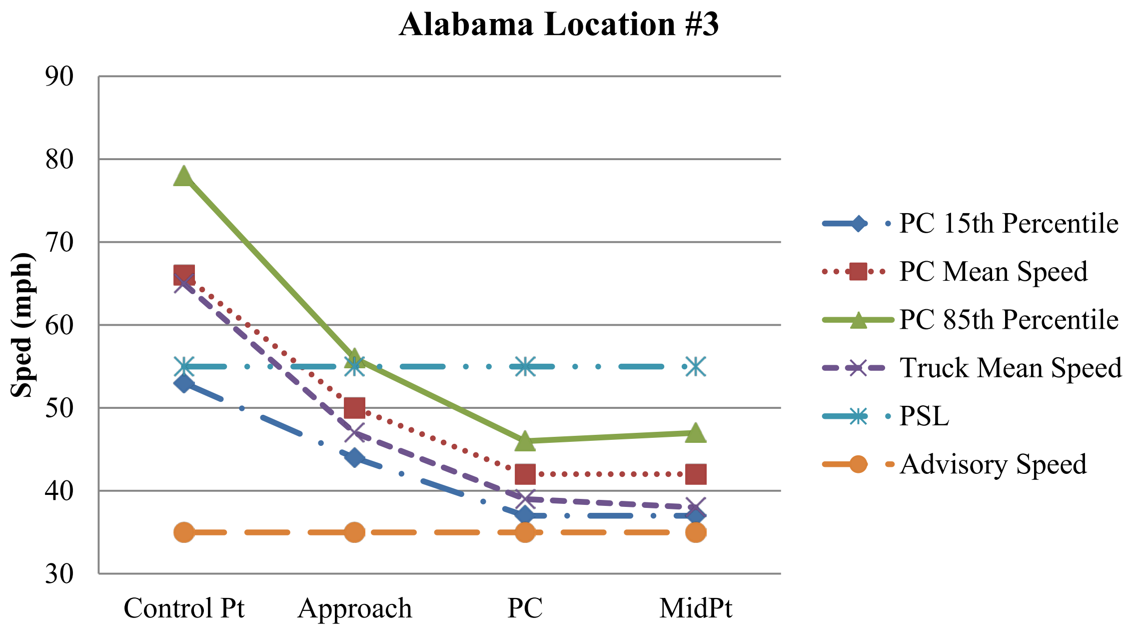


Figure 76. Graph. Graphical representation of speeds at Alabama Location #3.

Alabama Location #4

The direction of travel for the data collection was southbound, and the curve direction was to the right. The radius of curve and the curve length, calculated using Google Earth™, were 236.38 ft and 277.70 ft, respectively. There was a substantial cut slope on the inside of the curve, which limited horizontal sight distance along the curve. There was guardrail and also chevrons on the outside of the curve. The PSL was 35 mph. As Figure 77 shows, there was a turn warning sign (W1-1) with an advisory speed plaque (W13-1P) of 20 mph and flashing amber beacons above the sign located at the curve approach. The travel lanes were 10 ft wide, and there was no shoulder on either side of the road.

The research team collected speed data at the control point, 0.3 mi before the curve approach, which was 500 ft before the PC. The team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve. The other one was placed 440 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 77 shows the horizontal curve layout, along with the speed data-collection locations.

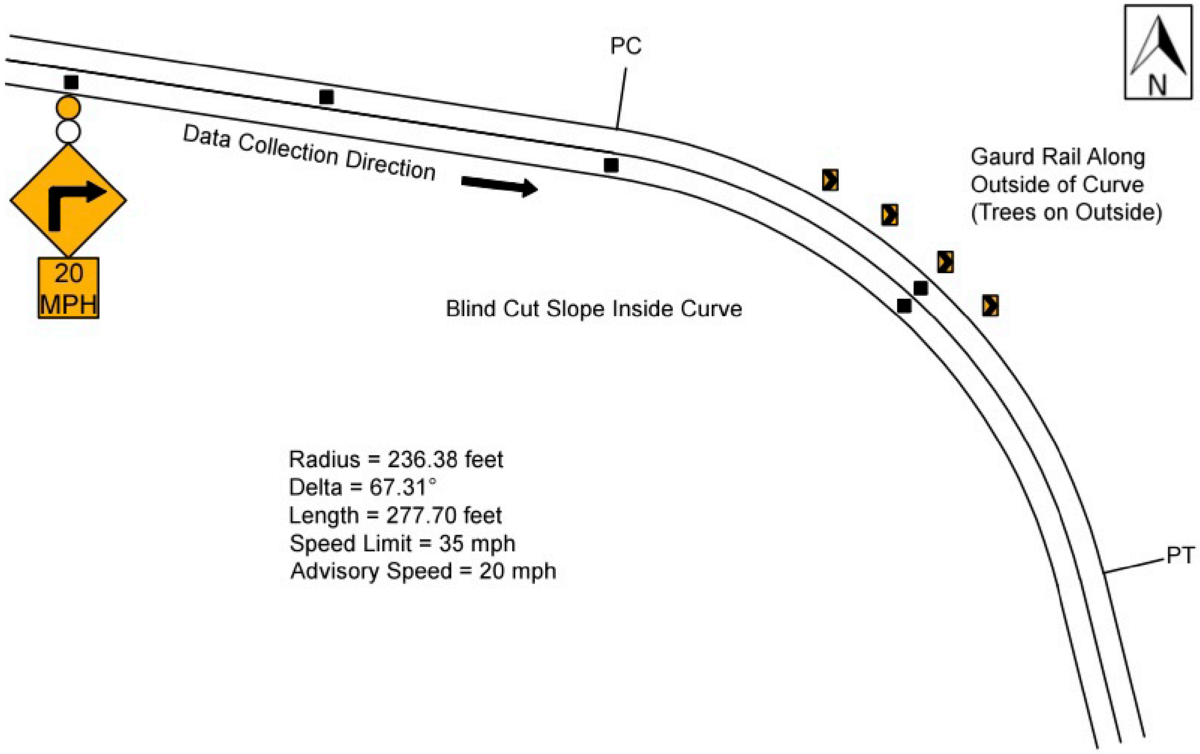


Figure 77. Diagram. Geometric layout of Alabama Location #4 (not to scale).

Table 48 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was only a statistically significant difference in speeds at the curve midpoint for passenger cars versus heavy trucks and daytime versus nighttime passenger cars.

Table 48. Alabama Location #4 before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	50.785	9.441	46.478	7.630	39.042	4.961	34.213	9.051	502	478	461
Passenger cars	50.660	9.345	46.506	7.597	39.100	4.908	**34.038**	9.068	480	459	443
Heavy trucks	53.500	11.237	45.864	8.481	37.632	6.103	**38.500**	7.649	22	19	18
Daytime passenger cars	50.656	8.996	46.149	8.725	39.462	4.987	**32.726**	10.008	195	182	179
Nighttime passenger cars	50.663	9.592	46.751	6.723	38.863	4.849	**34.928**	8.273	285	277	264
Opposed passenger cars	51.007	9.307	46.188	9.237	39.053	5.305	33.523	9.955	144	132	128
Unopposed passenger cars	50.512	9.371	46.643	6.786	39.119	4.746	34.248	8.690	336	327	315

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 78 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 78 also shows the mean speed for passenger cars and trucks within the study section. The figure shows the passenger car speeds and truck speeds decelerated substantially from the approach to the midpoint of the curve. The passenger car speeds and the truck mean speeds along the curve were closer to the 35 mph PSL than the advisory speed of 20 mph. The mean acceleration rate from the PC to the midpoint of the curve was -0.931 ft/s for passenger cars and -0.183 ft/s for trucks. A negative value indicates deceleration.

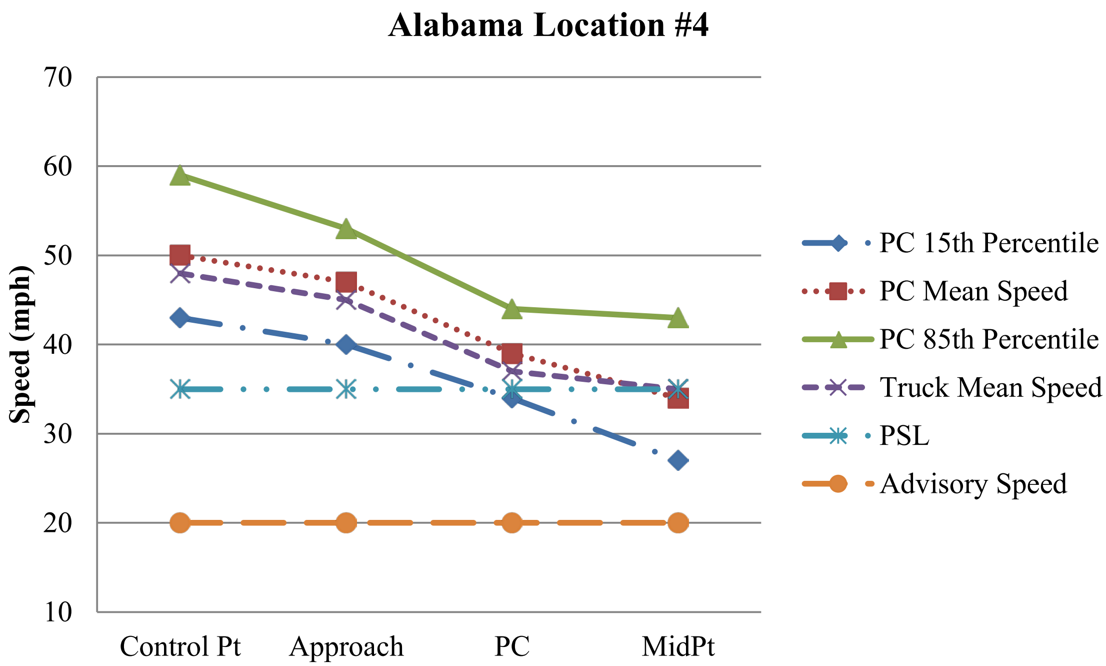


Figure 78. Graph. Graphical representation of speeds at Alabama Location #4.

Alabama Location #5

The direction of travel for the data collection was southbound, and the curve direction was to the right. The curve approach and curve were located on a downgrade. This curve to the right was part of a reverse curve, with the site for Alabama Location #6 located immediately downstream. The radius of curve and the curve length, calculated using Google Earth™, were 709.81 ft and 1,059.71 ft, respectively. The PSL was 40 mph. The travel lanes were 9 ft wide, and there was no shoulder on either side of the road.

The research team collected speed data at the control point, 1,000 ft before the curve approach, which was 250 ft before the PC. The team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 455 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 79 shows the horizontal curve layout, along with the speed data-collection locations.

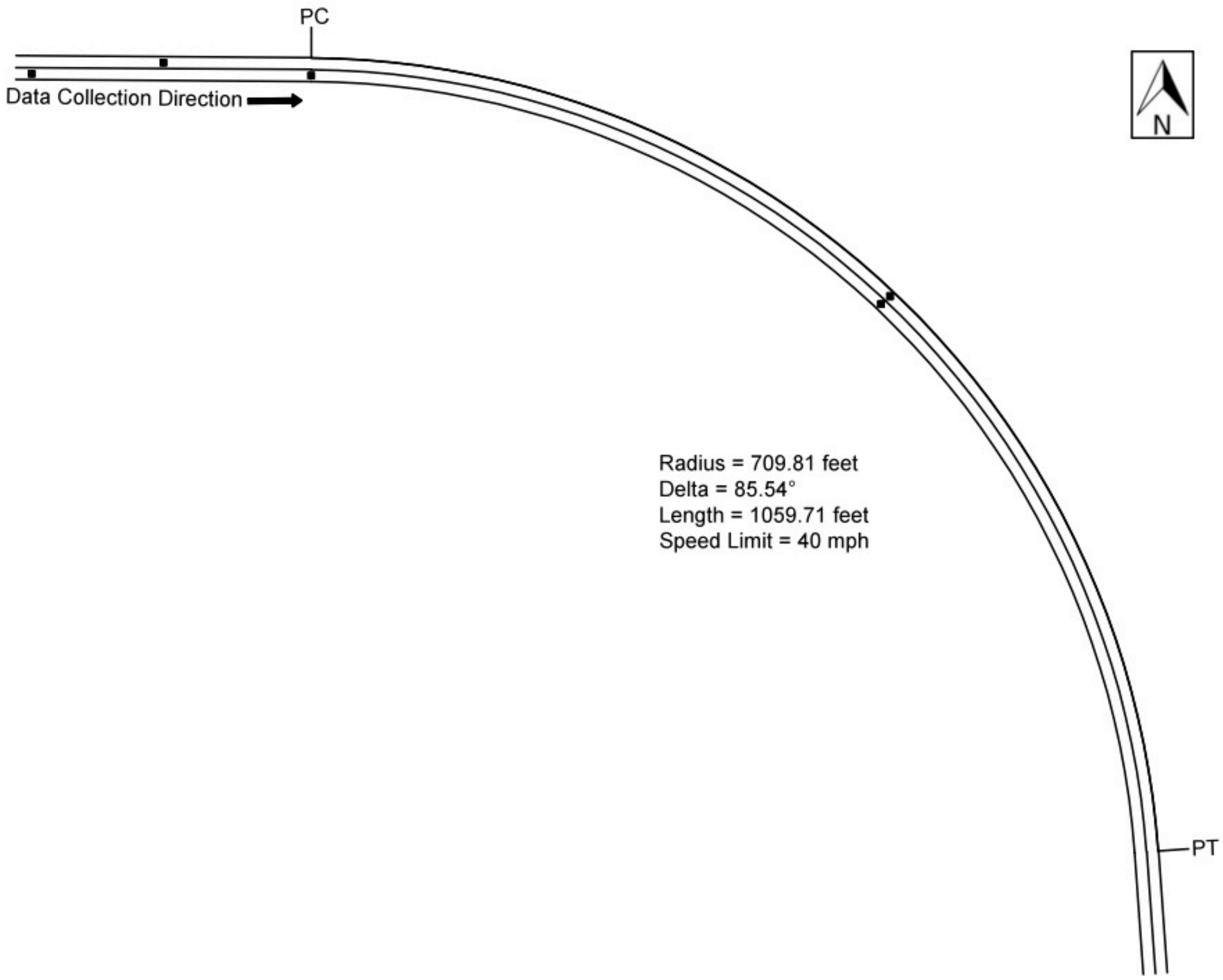


Figure 79. Diagram. Geometric layout of Alabama Location #5 (not to scale).

Table 49 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was a statistically significant difference in speeds at the control point, approach, and the curve midpoint for daytime versus nighttime passenger cars. There was also a statistically significant difference in speeds at the PC and midpoint of the curve for opposed versus unopposed passenger cars.

Table 49. Alabama Location #5 before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	41.944	7.212	43.742	8.327	45.781	7.524	39.810	6.842	233	64	221
Passenger cars	**42.299**	7.094	**44.054**	8.250	45.794	7.584	**40.028**	6.782	224	63	212
Heavy trucks	**33.111**	3.822	**36.000**	6.614	45.000	N/A	**34.667**	6.557	9	1	9
Daytime passenger cars	41.947	6.887	44.405	7.941	45.619	7.450	39.787	6.772	131	42	122
Nighttime passenger cars	42.796	7.383	43.559	8.686	46.143	8.021	40.356	6.821	93	21	90
Opposed passenger cars	40.235	7.242	41.353	11.045	**38.833**	7.333	**34.933**	7.045	17	6	15
Unopposed passenger cars	42.469	7.072	44.275	7.972	**46.526**	7.290	**40.416**	6.621	207	57	197

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint
N/A = Not Applicable

Figure 80 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 80 also shows the mean speed for trucks within the study section. As the figure shows, the speeds for passenger cars and the truck mean speeds gradually increased from the control point to the PC of the curve and decreased from the PC to the midpoint of the curve. The 85th percentile speeds of passenger cars along the curve were higher than the PSL of 40 mph. The mean acceleration rate from the PC to the midpoint of the curve was -7.817 ft/s for passenger cars and -0.369 ft/s for trucks. A negative value indicates deceleration.

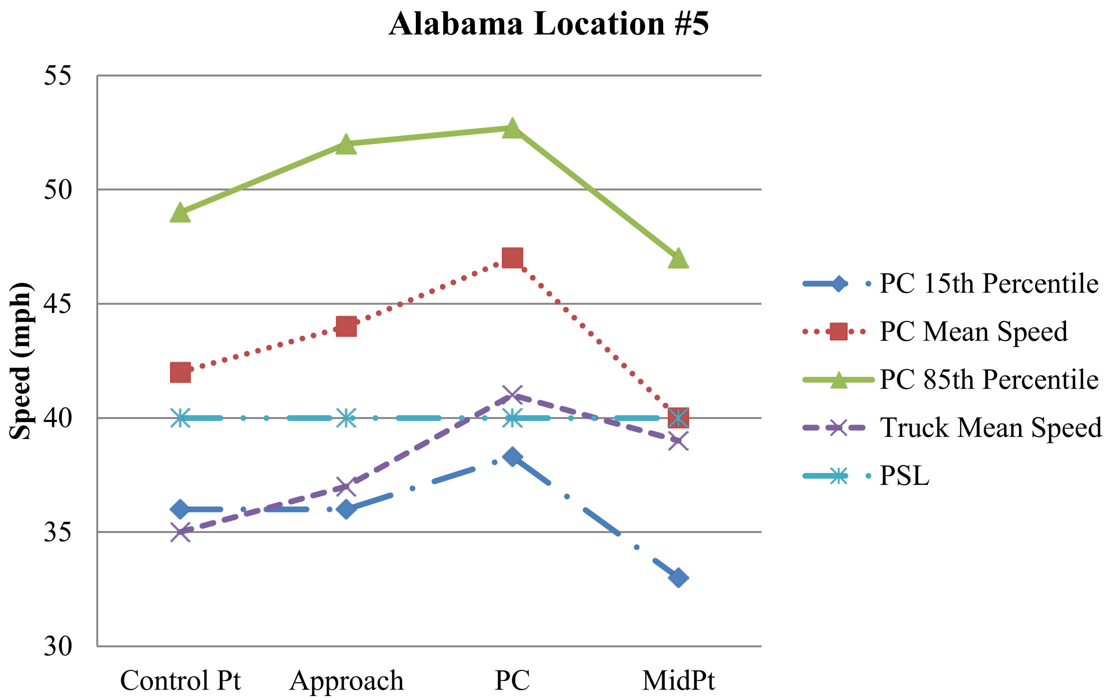


Figure 80. Graph. Graphical representation of speeds at Alabama Location #5.

Alabama Location #6

This curve to the right was part of a reverse curve associated with the site for Alabama Location #5 located immediately upstream. The direction of travel for the data collection was northbound, and the curve direction was to the right. The radius of curve and the curve length, calculated using Google Earth™, were 710.84 ft and 885.08 ft, respectively. At the midpoint of the curve, the curve starts to go upgrade with a driveway located on the outside of the curve. The PSL was 40 mph. The travel lanes were approximately 9 ft wide, and there was no shoulder on either side of the road.

The research team collected speed data at the control point, 750 ft before the curve approach, which was 500 ft before the PC. The team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 490 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 81 shows the horizontal curve layout, along with the speed data-collection locations.

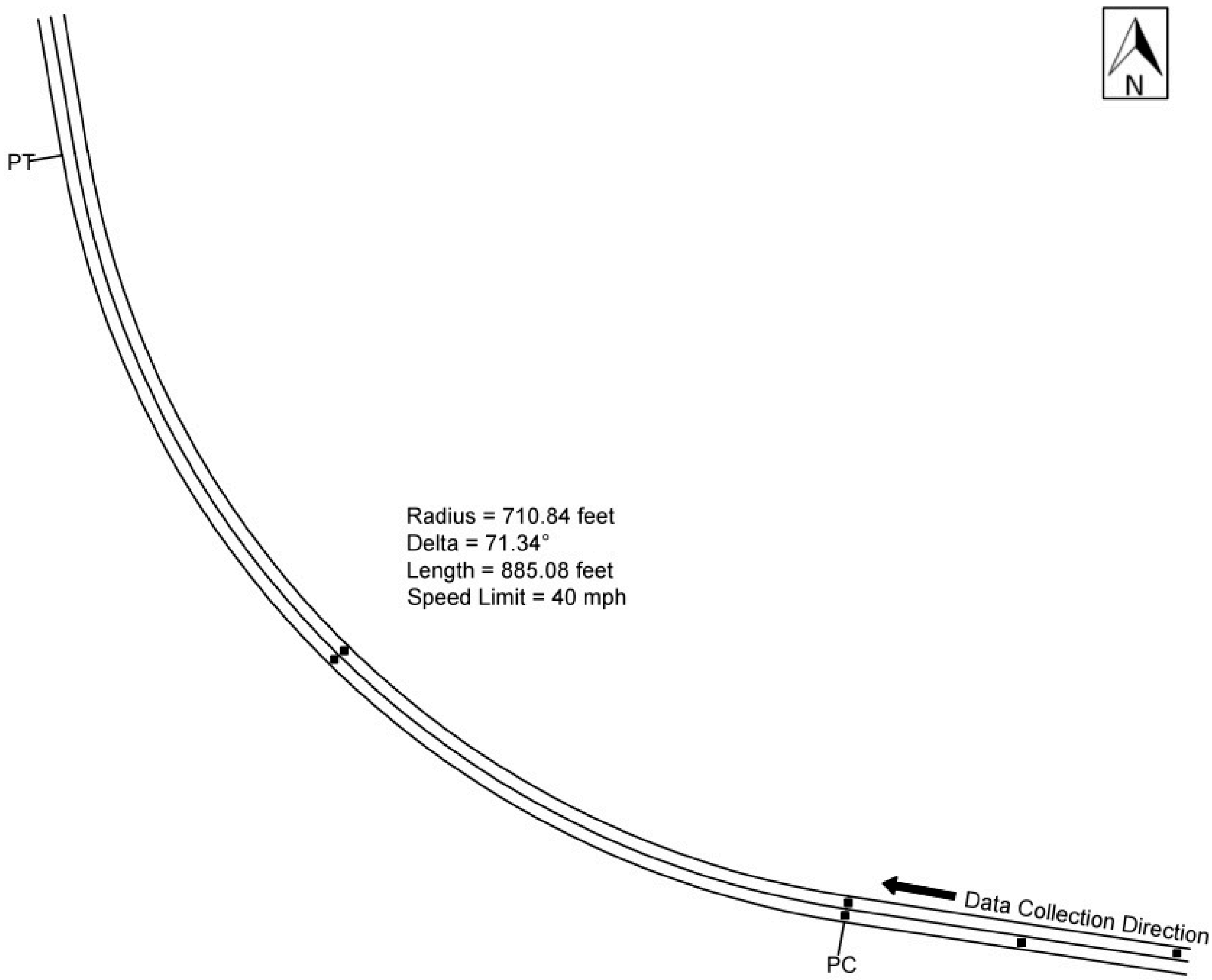


Figure 81. Diagram. Geometric layout of Alabama Location #6 (not to scale).

Table 50 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each curve location. For this treatment site, there was a statistically significant difference in speeds at the approach for passenger cars versus heavy trucks. There was also a statistically significant difference at the PC for opposed versus unopposed passenger cars.

Table 50. Alabama Location #6 before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	43.386	9.196	45.081	7.189	46.442	6.431	45.031	8.436	197	86	98
Passenger cars	43.370	9.277	**45.402**	6.928	46.447	6.469	45.128	8.568	189	85	94
Heavy trucks	43.750	7.498	**37.500**	9.457	46.000	N/A	42.750	4.272	8	1	4
Daytime passenger cars	43.462	9.112	45.400	6.716	46.586	6.489	44.317	9.561	130	58	63
Nighttime passenger cars	43.169	9.708	45.407	7.433	46.148	6.538	46.774	5.869	59	27	31
Opposed passenger cars	41.056	7.565	42.944	10.067	**43.400**	4.881	46.750	4.070	18	10	12
Unopposed passenger cars	43.614	9.425	45.661	6.499	**46.853**	6.571	44.890	9.032	171	75	82

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint
N/A = Not Applicable

Figure 82 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 82 also shows the mean speed for trucks within the study section. The figure shows that the 85th percentile and the mean speeds for passenger cars remain relatively stable from the control point to the midpoint of the curve. The 15th percentile speeds for passenger cars increased substantially from the control point to the approach of the curve and stabilized from the approach to the midpoint of the curve. The truck mean speeds were not consistent; the speeds decreased from the control point to the approach of the curve and increased from the approach to the PC and then decreased from the PC to the midpoint of the curve. The 85th percentile speeds and the mean speeds for passenger and the truck mean speeds along the curve were all higher than the PSL of 40 mph. The mean acceleration rate from the PC to the midpoint of the curve was 0.627 ft/s for passenger cars and -1.038 ft/s for trucks. A negative value indicates deceleration.

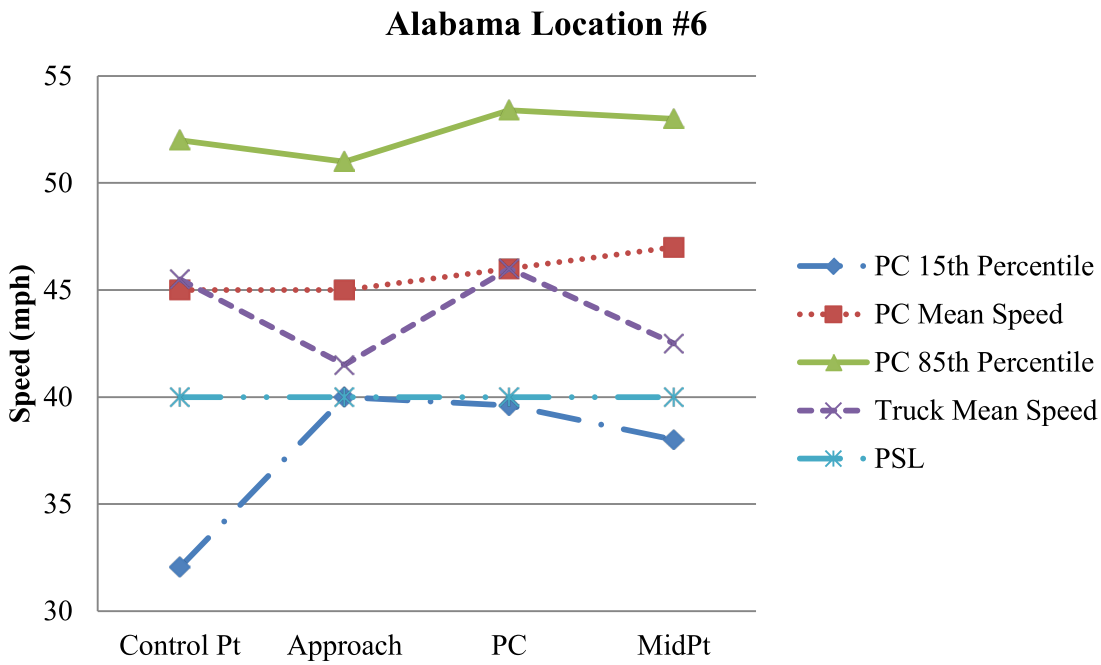


Figure 82. Graph. Graphical representation of speeds at Alabama Location #6.

Alabama Location #7

This curve to the right was part of a reverse curve, with a curve back to the left immediately after. The direction of travel for the data collection was southbound, and the curve direction was to the right. The white edge lines and yellow centerline were severely faded on the tangent and throughout the curve. The radius of curve and the curve length, calculated using Google Earth™, were 601.93 ft and 456.89 ft, respectively. The PSL was 35 mph. The travel lanes were 10 ft wide, and there was no shoulder on either side of the road. There was a driveway located on the right side of the road approximately 100 ft before the PC. As figure 83 shows, there was a SCHOOL BUS STOP AHEAD sign (S3-1), with a REDUCED SPEED AHEAD sign (R2-5) above it, located on the curve approach. There was also a winding road sign (W1-5) with a speed limit sign (R2-1) of 35 mph 250 ft before the PC.

The research team collected speed data at the control point, 0.3 mi before the curve approach, which was 500 ft before the PC. The team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 450 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 83 shows the horizontal curve layout, along with the speed data-collection locations.

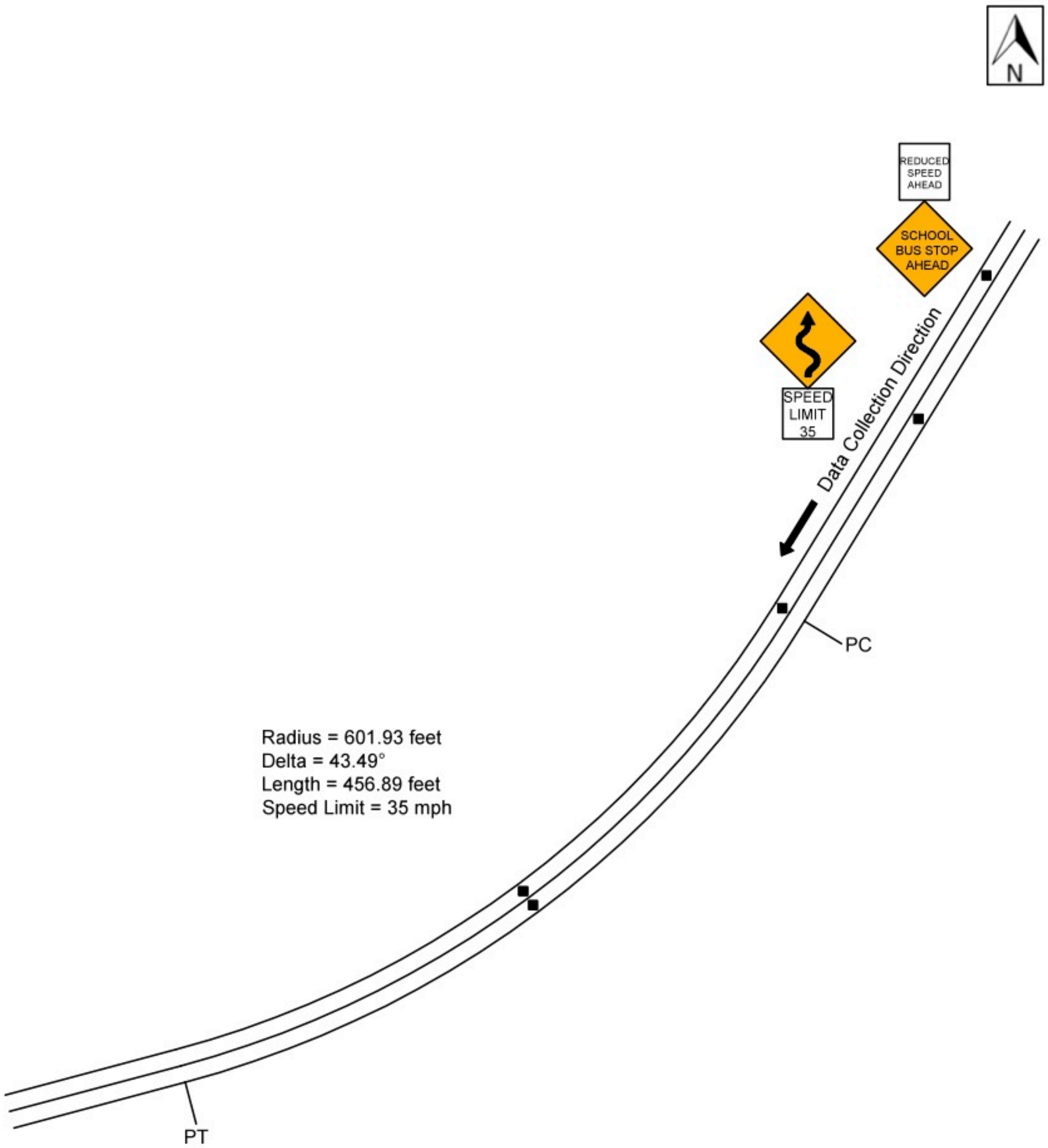


Figure 83. Diagram. Geometric layout of Alabama Location #7 (not to scale).

Table 51 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team also conducted simple *t*-tests for each of these comparisons at each curve location. For this treatment site, there was a statistically significant difference in speeds at the control point for passenger cars versus heavy vehicles. There was also a statistically significant difference in speeds at the approach and PC for daytime versus nighttime passenger cars.

Table 51. Alabama Location #7 before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	52.977	9.890	43.666	5.396	36.953	4.288	38.806	5.266	431	423	422
Passenger cars	**52.850**	9.896	43.653	5.388	36.973	4.305	38.755	5.239	421	413	412
Heavy trucks	**58.300**	8.433	44.200	6.033	36.100	3.604	40.900	6.208	10	10	10
Daytime passenger cars	53.000	9.668	**44.420**	5.224	**37.647**	4.536	39.188	5.310	193	190	191
Nighttime passenger cars	52.724	10.104	**43.004**	5.450	**36.399**	4.020	38.380	5.160	228	223	221
Opposed passenger cars	52.148	8.668	43.241	5.701	36.657	4.310	38.355	4.989	108	105	107
Unopposed passenger cars	53.093	10.287	43.796	5.277	37.081	4.306	38.895	5.325	313	308	305

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 84 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 84 also shows the mean speed for trucks within the study section. As the figure shows, the passenger car speeds and the truck mean speeds decelerated substantially from the control point to the PC of the curve. Passenger car speeds and the truck mean speeds remained relatively constant from the PC through the curve, averaging between 35 and 40 mph. The 85th percentile speeds and mean speeds of passenger cars and the truck mean speeds along the curve were higher than the PSL of 35 mph. The mean acceleration rate from the PC to the midpoint of the curve was 0.885 ft/s for passenger cars and 0.713 ft/s for trucks.

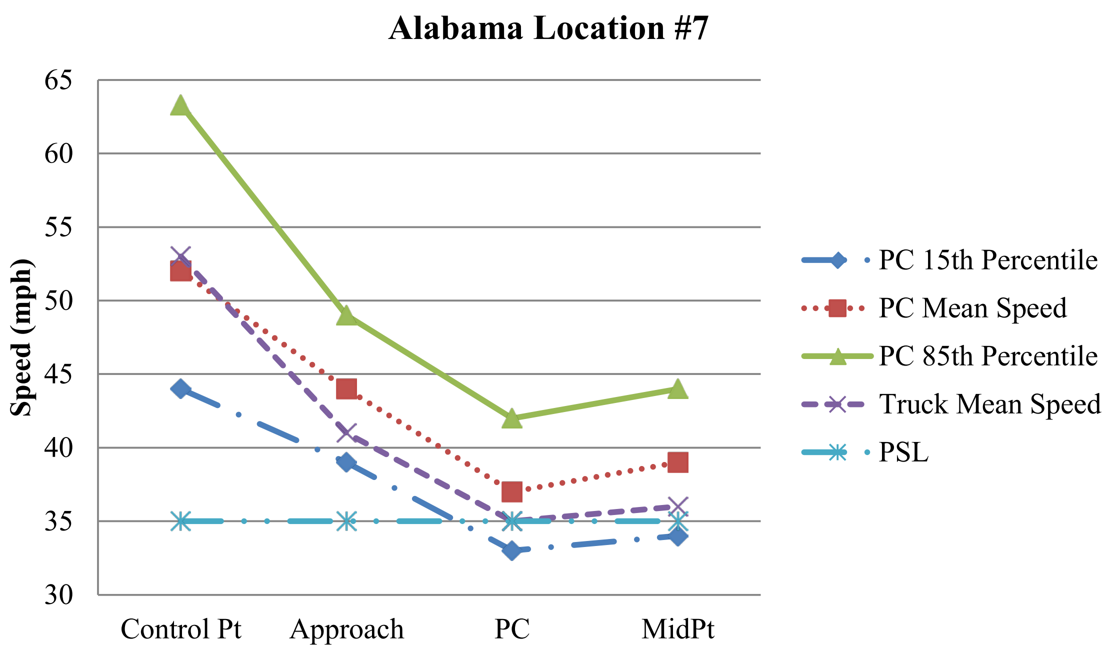


Figure 84. Graph. Graphical representation of speeds at Alabama Location #7.

Alabama Location #8

This curve was located approximately 1,000 ft downstream of Alabama Location #7 in the opposite direction. The direction of travel for the data collection was northbound, and the curve direction was to the right. There was a cut slope on the inside of the curve, which partially restricted horizontal sight distance entering the curve. The white edge lines and yellow centerline were severely faded on the tangent and throughout the curve. The radius of curve and the curve length, calculated using Google Earth, were 486.26 ft and 222.95 ft, respectively. The PSL was 35 mph. The travel lanes were 10 ft wide and there was no shoulder on either side of the road. There was a driveway on the outside of the curve at the midpoint. As figure 85 shows, there was a winding road sign (W1-5) with a speed limit sign (R2-1) of 35 mph located 700 ft before the curve approach. The end of a 280 ft concrete bridge crossing a ravine ended approximately 180 ft before the PC.

The research team collected speed data at the control point, 1,500 ft before the curve approach, which was 500 ft before the PC. The team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, The other one was placed 415 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 85 shows the horizontal curve layout, along with the speed data-collection locations.

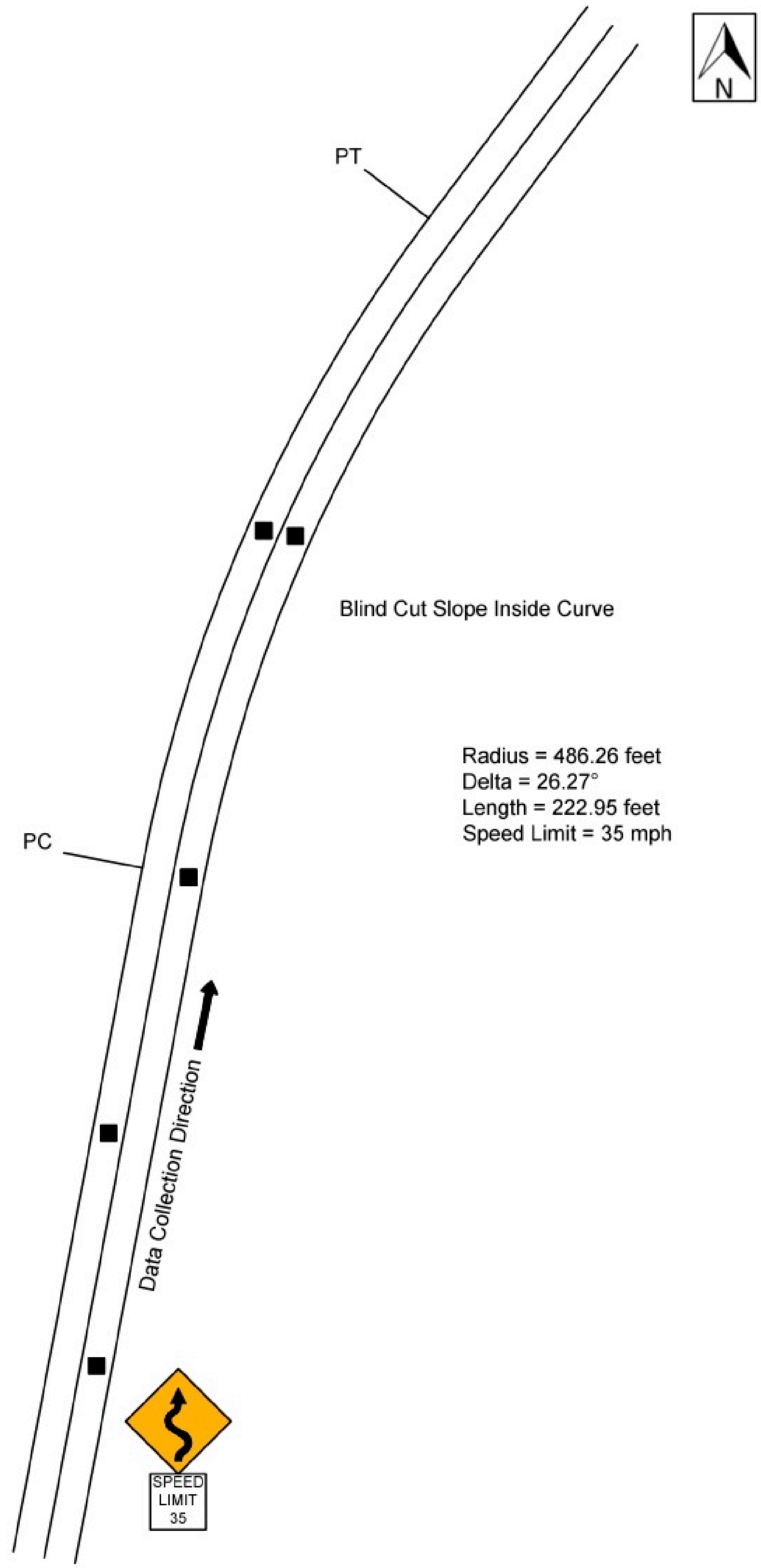


Figure 85. Diagram. Geometric layout of Alabama Location #8 (not to scale).

Table 52 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team again performed a simple *t*-test for each of these comparisons at each curve location. For this treatment site, there was a statistically significant difference in speeds at the PC and curve midpoint for passenger cars versus heavy vehicles. There was also a statistically significant difference in speeds at the control point for daytime versus nighttime passenger cars. There was also a statistically significant difference in speeds at the approach for opposed versus unopposed passenger cars.

Table 52. Alabama Location #8 before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	41.540	6.029	44.644	8.437	40.661	5.324	42.266	6.329	376	342	312
Passenger cars	41.566	6.053	44.672	8.317	**40.754**	5.326	**42.375**	6.337	366	334	304
Heavy trucks	40.600	5.254	43.600	12.624	**36.750**	3.655	**38.125**	4.612	10	8	8
Daytime passenger cars	**43.145**	5.833	44.612	8.251	40.842	4.928	42.401	5.908	165	146	137
Nighttime passenger cars	**40.269**	5.934	44.721	8.391	40.686	5.628	42.353	6.686	201	188	167
Opposed passenger cars	41.750	6.708	**42.841**	7.546	40.823	6.354	42.000	7.255	88	79	71
Unopposed passenger cars	41.507	5.842	**45.252**	8.477	40.733	4.979	42.489	6.042	278	255	233

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 86 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 86 also shows the mean speed for trucks within the study section. As the figure shows, the patterns of the speed changes were similar for the passenger car speeds and the truck mean speeds at this curve location. The speeds for passenger cars and the truck mean speeds increased from the control point to the approach of the curve, decreased from the approach to the PC of the curve, and then increased again from the PC to the midpoint of the curve. The speeds of passenger cars and the truck mean speeds along the curve were both higher than the PSL of 35 mph. The mean acceleration rate from the PC to the midpoint of the curve was 1.94 ft/s for passenger cars and -0.153 ft/s for trucks. A negative value indicates deceleration.

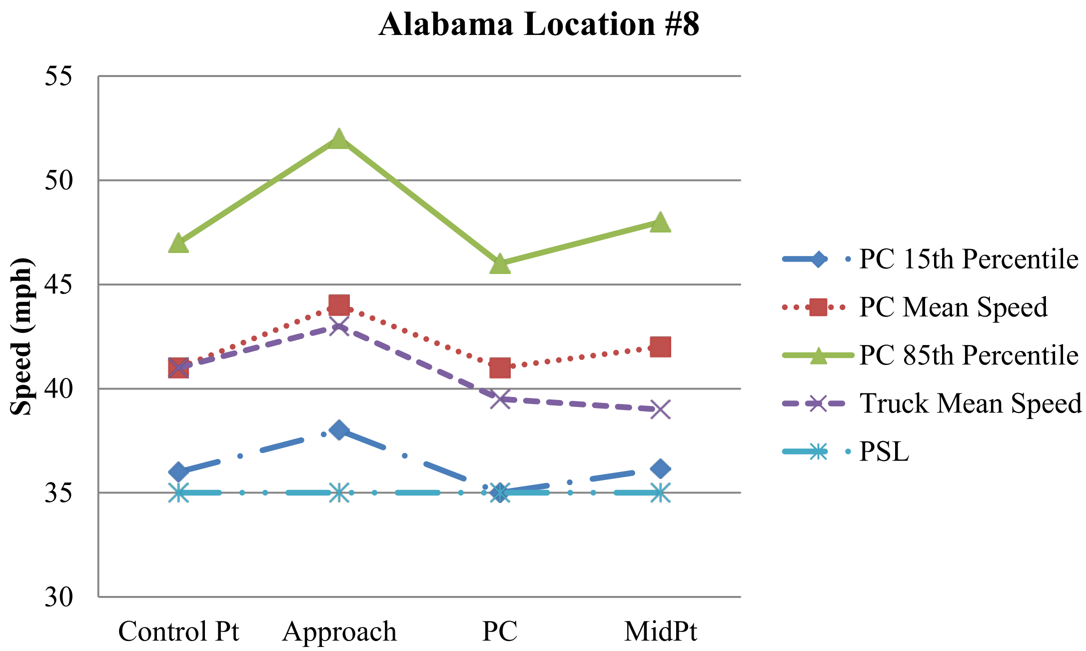


Figure 86. Graph. Graphical representation of speeds at Alabama Location #8.

Massachusetts

This section describes the characteristics of the seven treatment sites in Massachusetts.

Southbound Tucker Road, MA

The research team collected data on Tucker Road in Dartmouth, MA. The direction of travel for the data collection was southbound, and the curve direction was to the right. The radius of curve and the curve length, calculated using Google Earth™, were 1,098.66 ft and 816.67 ft, respectively. The PSL was 30 mph. The travel lanes were 13 ft wide, and there was no shoulder on either side of the road.

The team collected speed data at the control point, 0.2 mi before the curve approach, which was 319 ft before the PC. The research team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 347 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 87 shows the horizontal curve layout, along with the speed data-collection locations.

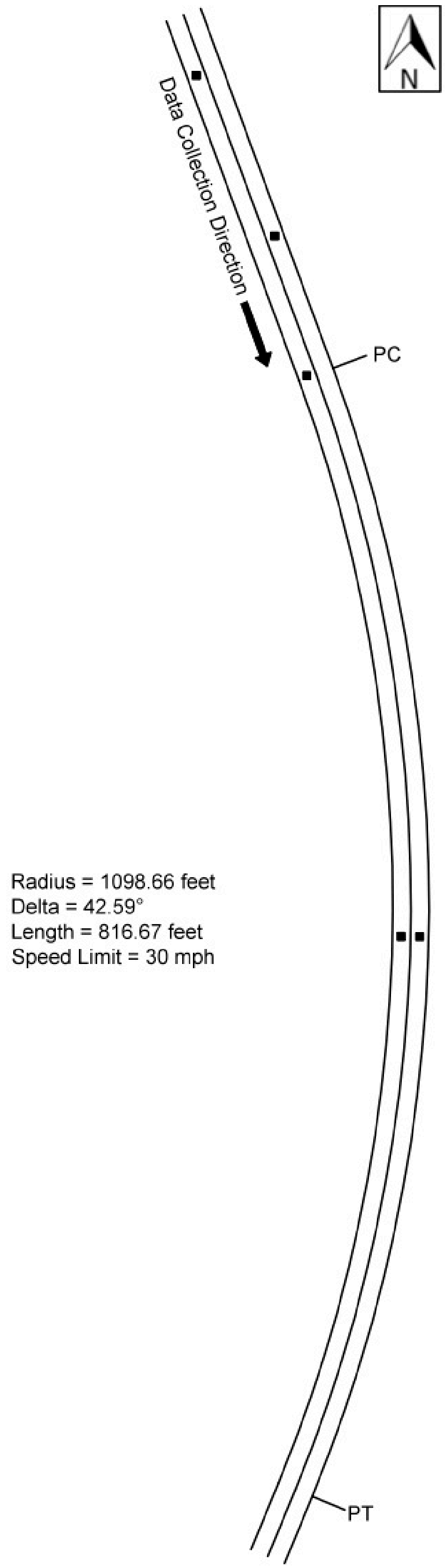


Figure 87. Diagram. Geometric layout of southbound Tucker Road (not to scale).

Table 53 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was a statistically significant difference in speeds at the control point and PC for passenger cars versus heavy trucks. There also was a statistically significant difference in speeds at the control point for daytime versus nighttime passenger cars, and also at the approach for opposed versus unopposed passenger cars.

Table 53. Southbound Tucker Road before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	44.458	7.485	36.492	5.228	35.223	4.467	35.329	5.014	577	539	514
Passenger cars	**44.466**	7.510	36.506	5.236	**35.241**	4.475	35.334	5.022	573	535	512
Heavy trucks	**43.250**	0.957	34.500	3.873	**32.750**	2.500	34.000	1.414	4	4	2
Daytime passenger cars	**45.136**	6.983	36.782	5.064	35.488	4.127	35.420	4.809	316	293	286
Nighttime passenger cars	**43.642**	8.048	36.167	5.431	34.942	4.855	35.226	5.289	257	242	226
Opposed passenger cars	44.306	7.210	**36.972**	5.543	35.272	3.792	35.036	5.203	252	235	224
Unopposed passenger cars	44.592	7.745	**36.140**	4.960	35.217	4.951	35.566	4.874	321	300	288

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 88 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 88 also shows the mean speed for trucks within the study section. As the figure shows, the passenger car speeds and truck speeds decelerated substantially from the control point to the approach of the curve. Both passenger car speeds and the truck mean speeds were relatively stable with minimal speed changes between the approach and the midpoint of the curve. The mean acceleration rate from the PC to the midpoint of the curve was 0.604 ft/s for passenger cars and 2.7 ft/s for trucks.

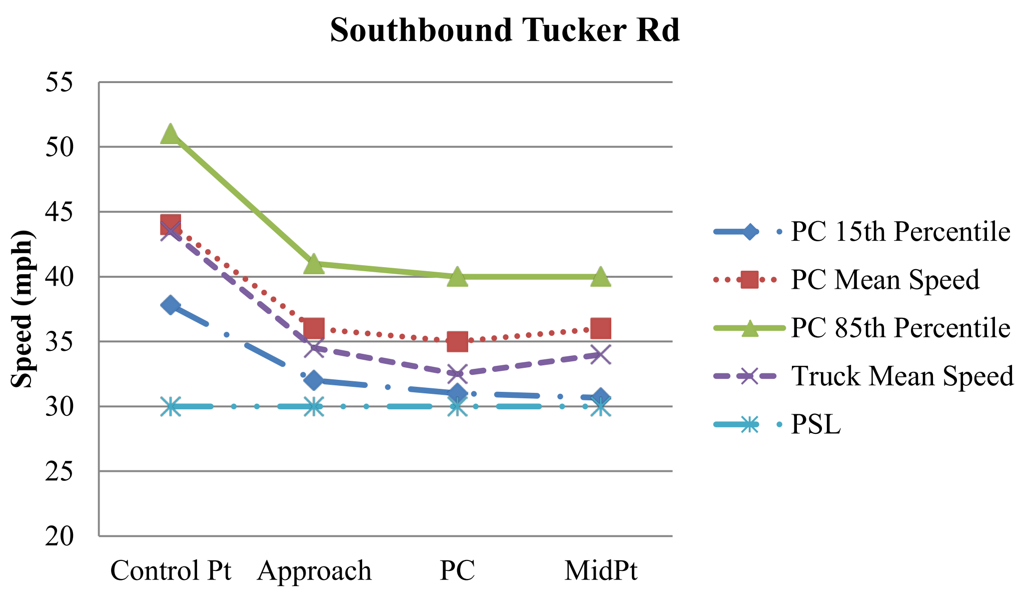


Figure 88. Graph. Graphical representation of speeds on southbound Tucker Road.

Northbound Tucker Road, MA

This curve was located approximately 0.9 mi upstream of the southbound Tucker Road curve. The direction of travel for the data collection was northbound, and the curve direction was to the left. This curve to the left was part of a reverse curve, with a curve back to the right. The radius of curve and the curve length, calculated using Google Earth™, were 568.76 ft and 293.24 ft, respectively. The PSL was 35 mph. The travel lanes were 12 ft wide, and there was no shoulder on either side of the road. There was a winding road sign (W1-5) located approximately 125 ft before the PC. An advisory speed plaque (W13-1P) of 25 mph was located approximately 100 ft upstream of the winding road sign. As figure 89 shows, there was one chevron on the outside of the curve at approximately the curve midpoint.

The research team collected speed data at the control point, 0.4 mi before the curve approach, which was 270 ft before the PC. The team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 262 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 89 shows the horizontal curve layout, along with the speed data-collection locations.

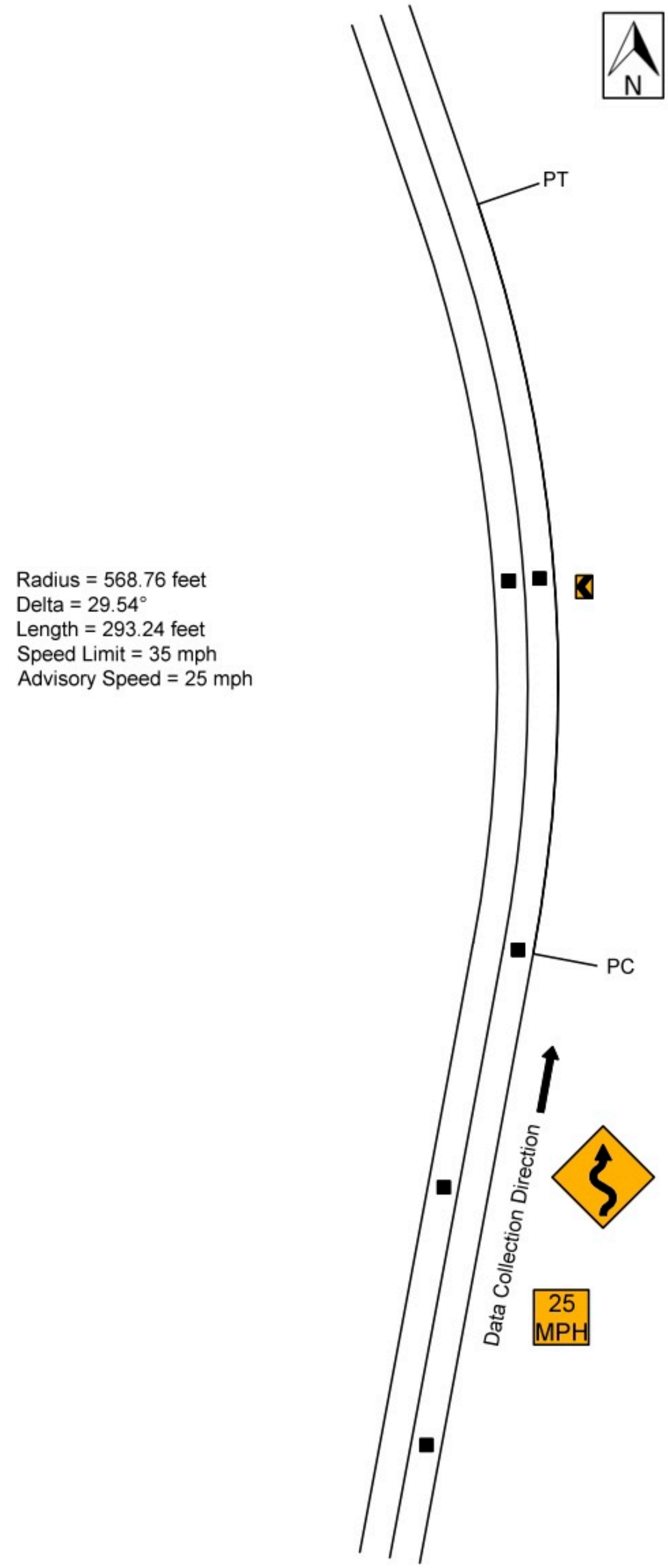


Figure 89. Diagram. Geometric layout of northbound Tucker Road (not to scale).

Table 54 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was a statistically significant difference in speeds at the control point, approach, and PC for passenger cars versus heavy trucks, with the heavy truck speed lower at these points. There also was a statistically significant difference in speeds at the PC and curve midpoint for daytime versus nighttime passenger cars. There also was a difference at the curve midpoint for opposed versus unopposed passenger cars.

Table 54. Northbound Tucker Road before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	33.895	4.361	37.609	4.247	33.381	3.820	35.507	4.852	573	565	458
Passenger cars	**33.919**	4.361	**37.633**	4.251	**33.403**	3.820	35.525	4.858	569	561	455
Heavy trucks	**30.500**	3.000	**34.250**	1.500	**30.250**	2.500	32.667	3.055	4	4	3
Daytime passenger cars	34.135	4.263	37.768	4.219	**33.683**	3.551	**36.145**	4.549	341	338	276
Nighttime passenger cars	33.596	4.495	37.430	4.300	**32.978**	4.167	**34.570**	5.168	228	223	179
Opposed passenger cars	33.681	4.172	37.453	4.207	33.369	3.635	**35.839**	4.758	351	347	280
Unopposed passenger cars	34.303	4.634	37.922	4.315	33.458	4.111	**35.023**	4.986	218	214	175

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 90 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 90 also shows the mean speed for trucks within the study section. As the figure shows, the patterns of the speed changes were similar for the passenger car speeds and the truck mean speeds at this curve location. The speeds for passenger cars and the truck mean speeds increased from the control point to the approach of the curve, decreased from the approach to the PC of the curve, and then increasing again from the PC to the midpoint of the curve. The speeds for passenger cars and the truck mean speeds along the curve were both higher than the PSL of 30 mph. The mean acceleration rate from the PC to the midpoint of the curve was 2.928 ft/s for passenger cars and 2.35 ft/s for trucks. A negative value indicates deceleration.

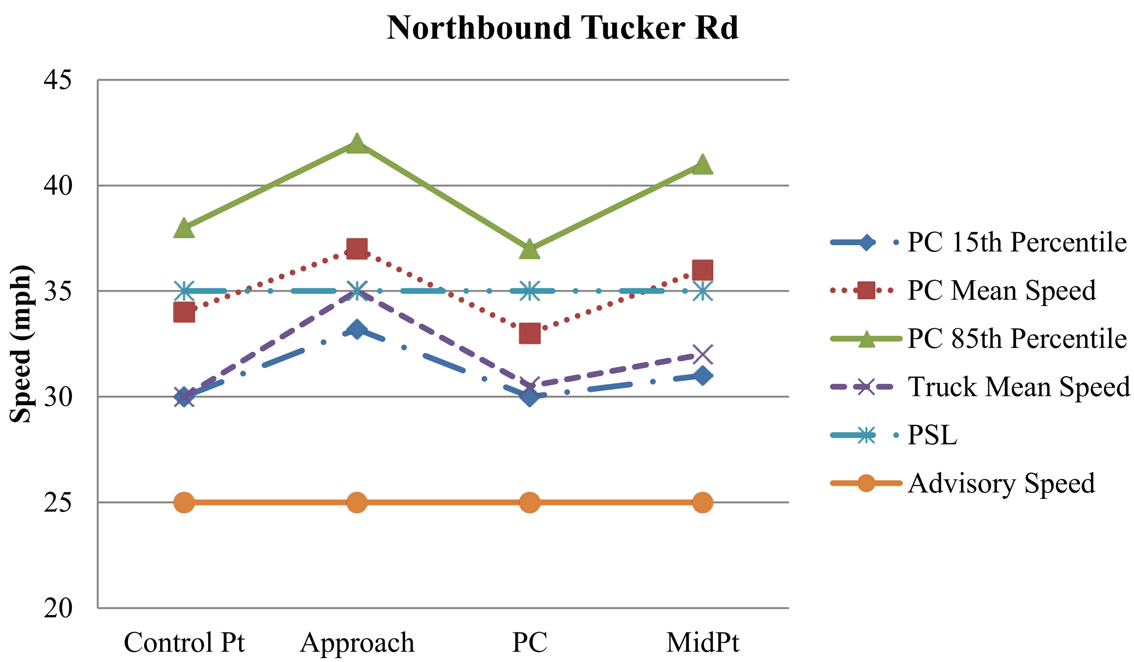


Figure 90. Graph. Graphical representation of speeds on northbound Tucker Road.

Southbound Reed Road, MA

The research team collected data on Reed Road in Dartmouth, MA. The direction of travel for the data collection was southbound and the curve direction was to the left. The radius of curve and the curve length, calculated using Google Earth™, were 683.31 ft and 458.56 ft, respectively. The PSL was 40 mph until 150 ft before the curve approach, where it changed to 25 mph. The travel lanes were 13 ft wide, and there was a 4-ft shoulder on both sides of the road. As figure 91 shows, there was a curve warning sign (W1-2) located between the curve approach and the PC. There were also three chevrons on the outside of the curve.

The team collected speed data at the control point, 0.3 mi before the curve approach, which was 431 ft before the PC. The research team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 311 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 91 shows the horizontal curve layout, along with the speed data-collection locations.

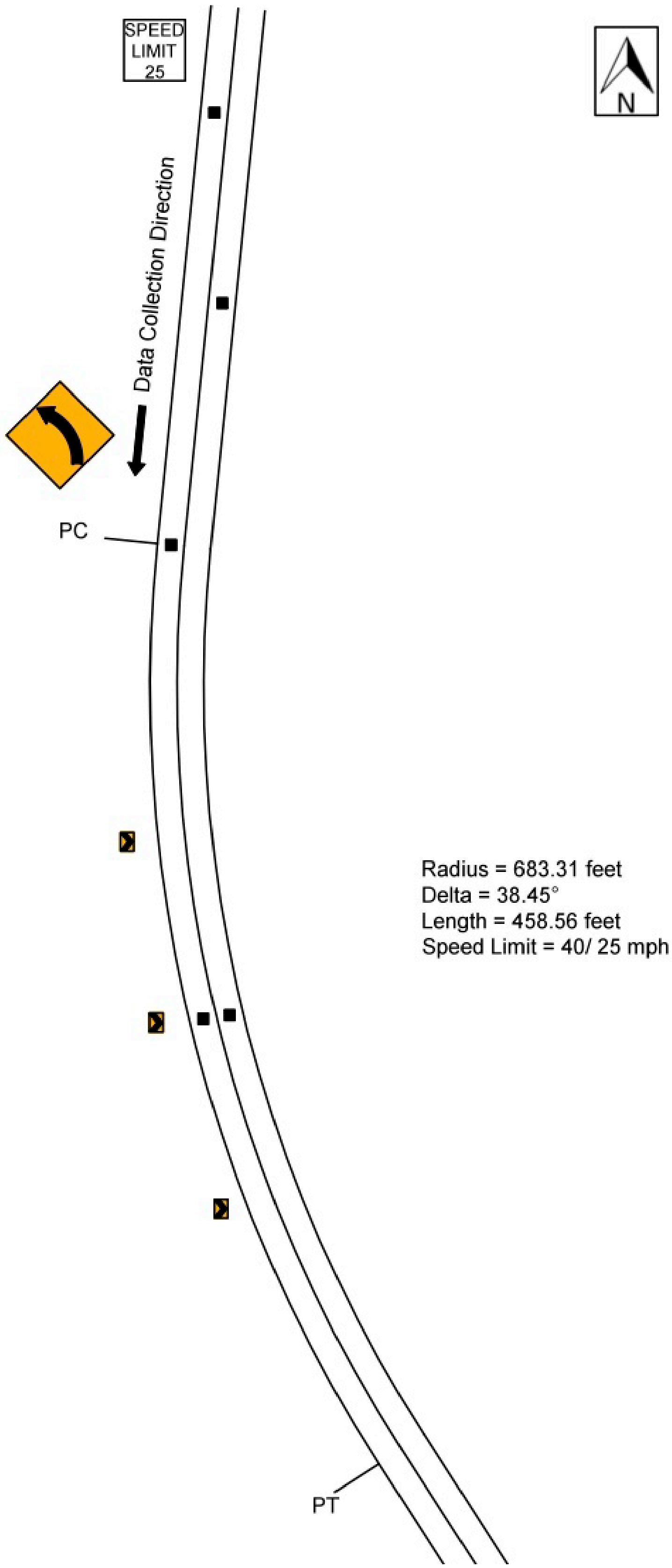


Figure 91. Diagram. Geometric layout of southbound Reed Road (not to scale).

Table 55 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was only a statistically significant difference in speeds at the control point and approach speed for passenger cars versus heavy trucks, with the heavy truck speed lower at these points.

Table 55. Southbound Reed Road before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All Observations	39.966	5.426	39.382	6.015	38.826	4.288	39.363	4.990	775	482	590
Passenger cars	**40.036**	5.395	**39.450**	5.985	38.811	4.227	39.416	4.945	756	471	575
Heavy trucks	**37.211**	6.070	**36.684**	6.758	39.455	6.654	37.333	6.355	19	11	15
Daytime passenger cars	39.754	5.282	39.249	6.262	38.558	3.773	39.692	4.767	337	224	247
Nighttime passenger cars	40.263	5.481	39.611	5.754	39.040	4.596	39.207	5.072	419	247	328
Opposed passenger cars	39.971	5.365	39.753	6.653	38.669	3.911	39.469	4.916	377	242	275
Unopposed passenger cars	40.100	5.432	39.148	5.227	38.961	4.542	39.367	4.979	379	229	300

Bold indicates significance at 95 percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 92 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 92 also shows the mean speed for trucks within the study section. The figure shows that the speeds for passenger cars remained relatively stable along the curve, and the speed changes were minimal. The truck mean speeds increased slightly from the control point to the PC and decreased slightly from the PC to the midpoint of the curve. The passenger car speeds and the truck mean speeds along the curve were higher than the PSL of 25 mph, averaging between 37 to 39 mph. The mean acceleration rate from the PC to the midpoint of the curve was -0.092 ft/s for passenger cars and -1.863 ft/s for trucks. A negative value indicates deceleration.

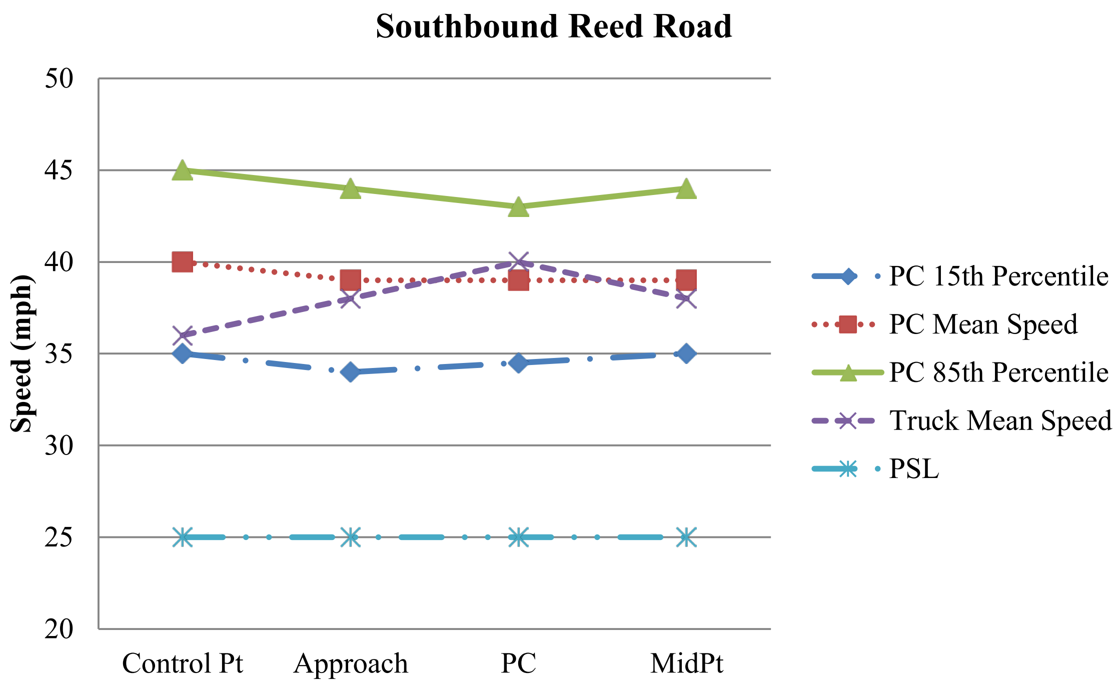


Figure 92. Graph. Graphical representation of speeds on southbound Reed Road.

Southbound New Boston Road, MA

The research team collected data on New Boston Road in Fairhaven, MA. The direction of travel for the data collection was southbound, and the curve direction was to the left. The radius of curve and the curve length, calculated using Google Earth™, were 592.52 ft and 226.37 ft, respectively. The PSL was 35 mph. The travel lanes were 11 ft wide, and there was no shoulder on either side of the road.

The team collected speed data at the control point, 0.2 mi before the curve approach, which was 530 ft before the PC. The research team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 460 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 93 shows the horizontal curve layout, along with the speed data-collection locations.

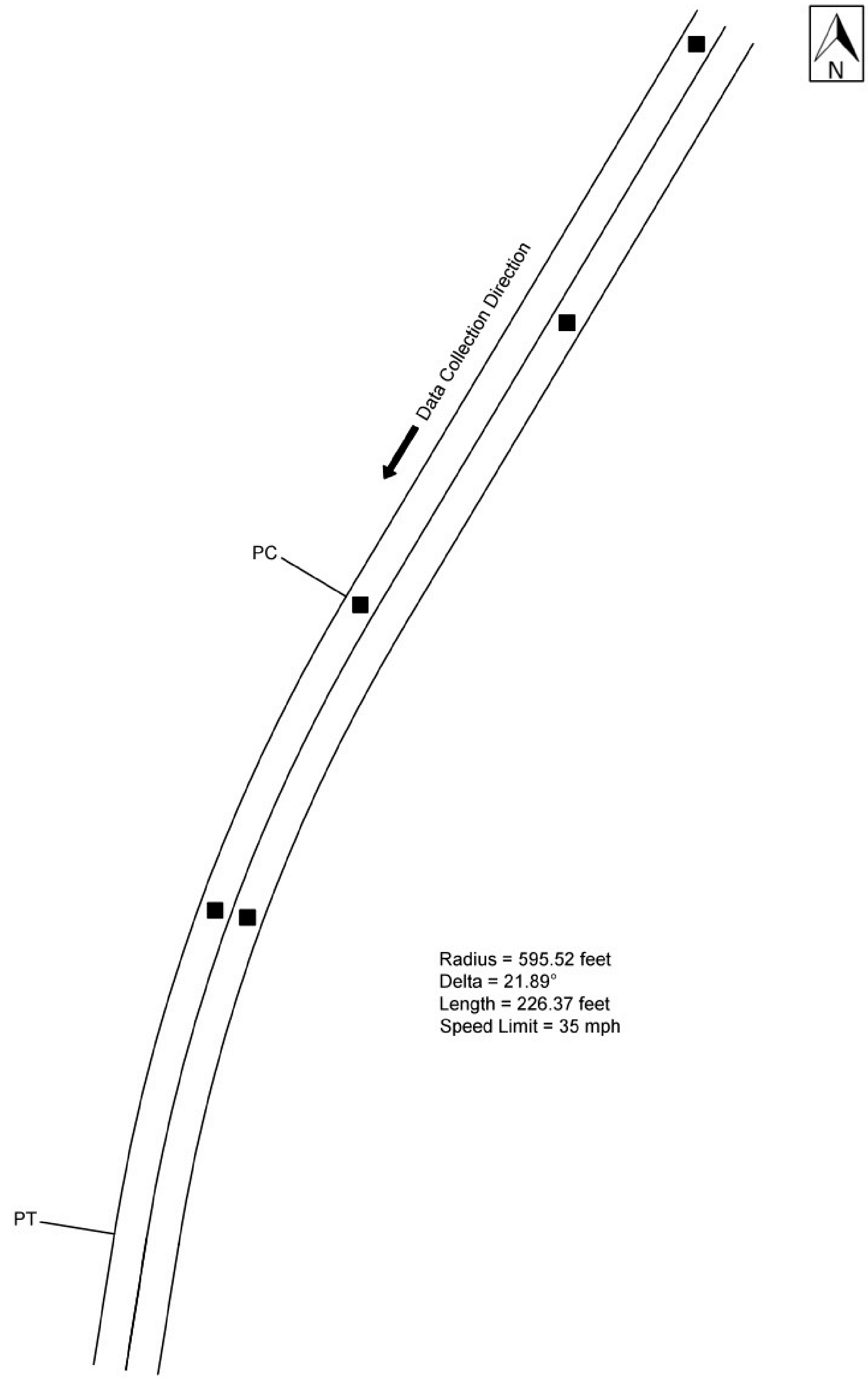


Figure 93. Diagram. Geometric layout of southbound New Boston Road (not to scale).

Table 56 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was a statistically significant difference in speeds at curve midpoint for passenger cars versus heavy trucks. There also was a statistically significant difference in speeds at the control point and approach for opposed versus unopposed passenger cars.

Table 56. Southbound New Boston Road before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	37.004	6.532	37.704	5.967	37.489	5.607	36.624	6.206	257	229	229
Passenger cars	36.968	6.565	37.766	5.980	37.465	5.636	**36.656**	6.224	252	226	227
Heavy trucks	38.800	4.712	34.600	4.722	39.333	2.082	**33.000**	0.000	5	3	2
Daytime passenger cars	36.690	5.930	37.655	5.583	37.576	4.603	36.760	5.551	113	99	100
Nighttime passenger cars	37.194	7.052	37.856	6.302	37.378	6.343	36.575	6.728	139	127	127
Opposed passenger cars	**35.638**	5.307	**36.259**	5.872	36.788	5.410	35.962	6.145	58	52	52
Unopposed passenger cars	**37.366**	6.859	**38.216**	5.953	37.667	5.702	36.863	6.250	194	174	175

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 94 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 94 also shows the mean speed for trucks within the study section. The figure shows that the speeds for passenger cars remained relatively stable along the curve, and the speed changes were minimal. The truck mean speeds decreased slightly between the control point and the approach of the curve, increased significantly from the approach to the PC of the curve, and then decreased substantially from the PC to the midpoint of the curve. The 85th percentile speeds and the mean speeds for passenger cars along the curve were higher than the PSL of 35 mph. The mean acceleration rate from the PC to the midpoint of the curve was -0.490 ft/s for passenger cars and -0.960 ft/s for trucks. A negative value indicates deceleration.

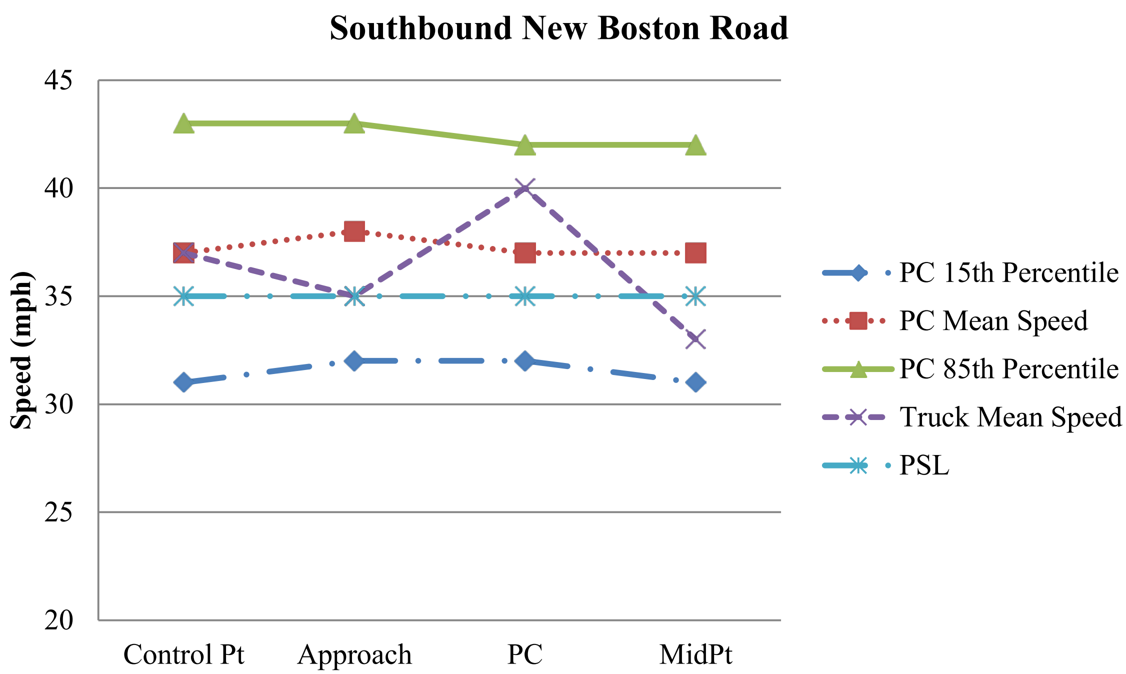


Figure 94. Graph. Graphical representation of speeds on southbound New Boston Road.

Northbound New Boston Road, MA

This curve was located approximately 800 ft downstream of the southbound New Boston Road curve. The direction of travel for the data collection was northbound, and the curve direction was to the right. The radius of curve and the curve length, calculated using Google Earth™, were 470.83 ft and 186.21 ft, respectively. The PSL was 35 mph. The travel lanes were 11 ft wide, and there was no shoulder on either side of the road.

The research team collected speed data at the control point, 0.4 mi before the curve approach, which was 470 ft before the PC. The team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 464 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 95 shows the horizontal curve layout, along with the speed data-collection locations.

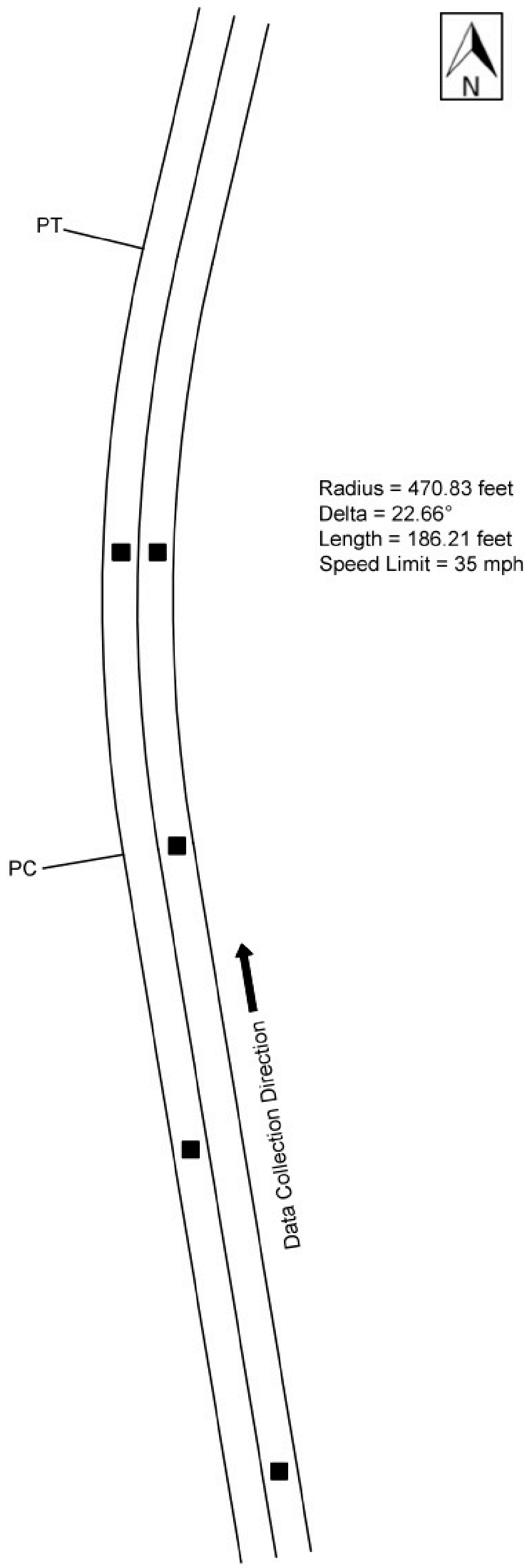


Figure 95. Diagram. Geometric layout of northbound New Boston Road (not to scale).

Table 57 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was a statistically significant difference in speeds at the control point and approach for passenger cars versus heavy trucks, with the truck speed lower at these points. There also was a statistically significant difference in speeds at the curve midpoint for daytime versus nighttime passenger cars.

Table 57. Northbound New Boston Road before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	39.796	5.704	36.876	5.691	39.500	4.955	33.742	6.553	225	114	209
Passenger cars	**39.857**	5.680	**36.928**	5.686	39.558	4.939	33.720	6.563	223	113	207
Heavy trucks	**33.000**	5.657	**31.000**	2.828	33.000	N/A	36.000	7.071	2	1	2
Daytime passenger cars	40.074	5.352	37.475	5.801	40.066	4.694	**34.627**	6.531	122	61	110
Nighttime passenger cars	39.594	6.070	36.267	5.499	38.962	5.194	**32.691**	6.478	101	52	97
Opposed passenger cars	40.119	4.835	36.810	4.830	39.714	3.989	32.789	4.911	42	21	38
Unopposed passenger cars	39.796	5.870	36.956	5.878	39.522	5.149	33.929	6.874	181	92	169

Bold indicates significance at 95 percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint
N/A = Not Applicable

Figure 96 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 96 also shows the mean speed for trucks within the study section. The figure shows that the speeds for passenger cars remained relatively stable from the control point to the PC of the curve, but decreased substantially from the PC to the midpoint of the curve. The truck mean speeds decreased slightly from the control point to the approach of the curve and increased slightly from the approach through the curve. The 85th percentile speeds for passenger cars along the curve were higher than the PSL of 35 mph. The mean acceleration rate from the PC to the midpoint of the curve was -5.478 ft/s for passenger cars and -0.2 ft/s for trucks. A negative value indicates deceleration.

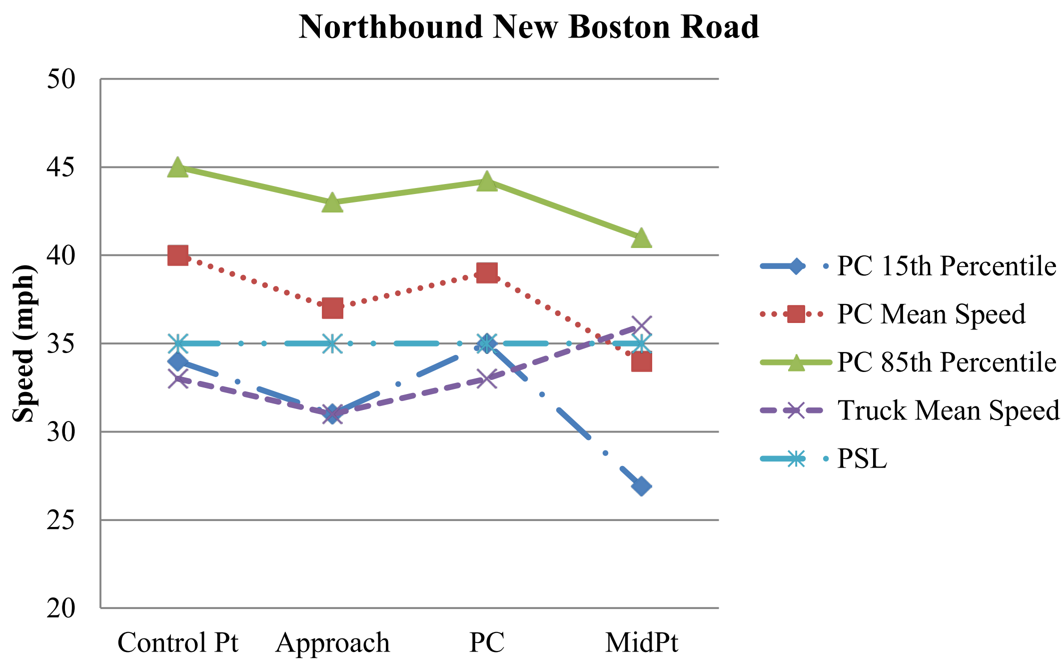


Figure 96. Graph. Graphical representation of speeds on northbound New Boston Road.

Southbound Braley Hill Road, MA

The research team collected data on Braley Hill Road in Rochester, MA. The direction of travel for the data collection was southbound, and the curve direction was to the right. The radius of curve and the curve length, calculated using Google Earth™, were 695.24 ft and 672.97 ft, respectively. There were trees and other growth on the inside of the curve, which partially restricted horizontal sight distance. The PSL was 30 mph. The travel lanes were 11 ft wide, and there was no shoulder on either side of the road. As figure 97 shows, there was a Dangerous Curves sign with a 30-mph speed limit sign (R2-1) located between the curve approach and the PC.

The research team collected speed data at the control point, 0.4 mi before the curve approach, which was 461 ft before the PC. The team also collected speed data at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 410 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 97 shows the horizontal curve layout, along with the speed data-collection locations.

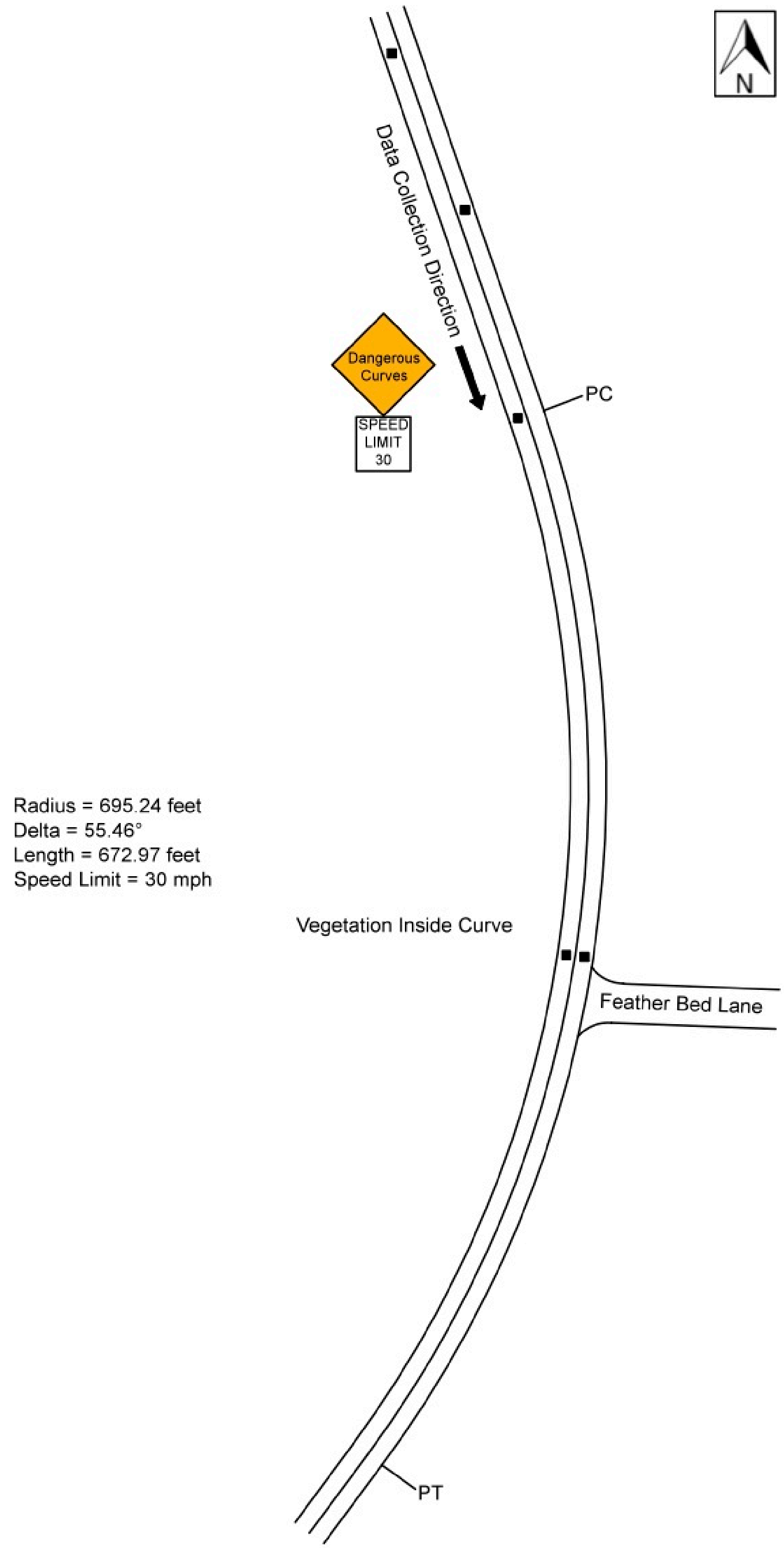


Figure 97. Diagram. Geometric layout of southbound Braley Hill Road (not to scale).

Table 58 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was a statistically significant difference in speeds at the control point and approach for passenger cars versus heavy vehicles, with the truck speed lower at these points. There also was a statistically significant difference in speeds at the control point, approach, and curve midpoint for daytime versus nighttime passenger cars, with the nighttime speeds higher at these points. There also was a statistically significant difference in speeds at the curve midpoint for opposed versus unopposed passenger cars.

Table 58. Southbound Braley Hill Road before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	39.360	5.097	38.508	5.835	34.585	4.791	36.365	6.947	317	294	252
Passenger cars	**39.472**	5.136	**38.600**	5.872	34.665	4.693	36.502	6.905	305	284	243
Heavy trucks	**36.500**	2.844	**36.167**	4.345	32.300	6.977	32.667	7.483	12	10	9
Daytime passenger cars	**38.640**	4.423	**37.953**	4.997	34.433	4.591	**35.697**	7.398	150	141	122
Nighttime passenger cars	**40.277**	5.640	**39.226**	6.566	34.895	4.797	**37.314**	6.297	155	143	121
Opposed passenger cars	39.211	4.485	37.958	5.247	34.758	4.736	**35.185**	6.355	71	66	54
Unopposed passenger cars	39.551	5.324	38.795	6.046	34.638	4.691	**36.878**	7.025	234	218	189

Bold indicates significance at 95 percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 98 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 98 also shows the mean speed for trucks within the study section. The figure shows that the 85th percentile speeds and the mean speeds for passenger cars decreased slightly from the control point to the PC and then increased from the PC to the midpoint of the curve. The 15th percentile speeds of passenger car decreased slightly along the curve. The truck mean speeds increased slightly between the control point and the approach of the curve, decreased from the approach to the PC of the curve, and then increased significantly from the PC to the midpoint of the curve. The passenger car speeds and the truck mean speeds along the curve were higher than the PSL of 30 mph. The mean acceleration rate from the PC to the midpoint of the curve was 3.544 ft/s for passenger cars and 4.908 ft/s for trucks.

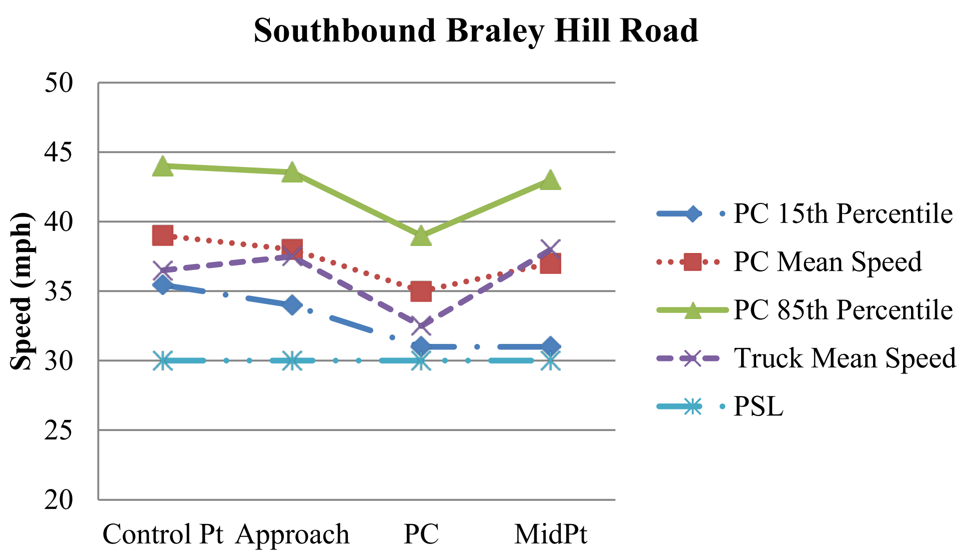


Figure 98. Graph. Graphical representation of speeds on southbound Braley Hill Road.

Northbound Braley Hill Road, MA

The northbound Braley Hill Road curve was located approximately 0.4 mi downstream of the southbound Braley Hill Road curve. The curve is located at the start of a broken back curve and compound curve. The direction of travel for the data collection was northbound, and the curve direction was to the right. The radius of curve and the curve length, calculated using Google Earth™, were 2,097.07 ft and 661.74 ft, respectively. There were trees and other growth approximately 5 to 10 ft off the inside of the curve, but horizontal sight distance was not restricted because of the large radius. The PSL was 40 mph. The travel lanes were 10 ft wide, and there was no shoulder on either side of the road. There were driveways spaced approximately 100 to 400 ft on both sides of the road on the curve approach and on the left side throughout the curve.

The research team collected speed data at the control point, 0.4 mi before the curve approach, which was 575 ft before the PC. Speed data were also collected at the PC and midpoint of the curve. One sensor in the opposing direction of travel was placed at the midpoint of the curve, and the other one was placed 415 ft downstream of the first sensor to determine whether vehicles were present in the opposing lane. Figure 99 shows the horizontal curve layout, along with the speed data-collection locations.

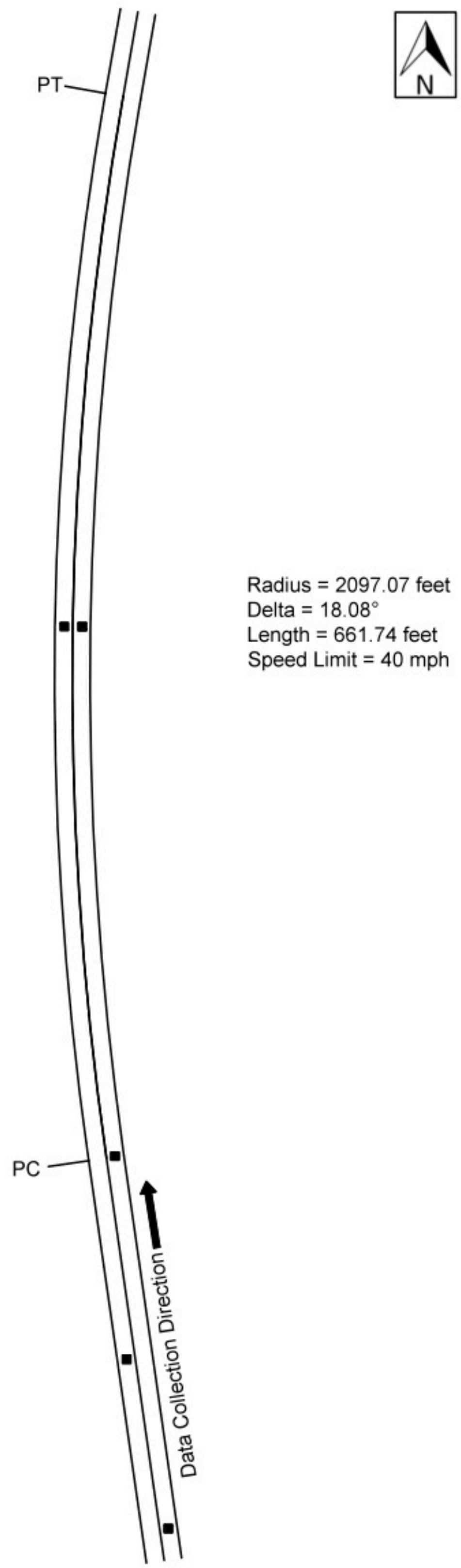


Figure 99. Diagram. Geometric layout of northbound Braley Hill Road (not to scale).

Table 59 shows the speed data for the before period. The data are shown as all observations, passenger car versus heavy trucks, daytime versus nighttime speeds for passenger cars, and opposed versus unopposed passenger cars. All of the comparisons exclude vehicles with high accelerations. The research team performed a simple *t*-test for each of these comparisons at each of the curve locations. For this treatment site, there was a statistically significant difference in speeds at the approach, PC, and curve midpoint for passenger cars versus heavy vehicles, with the truck speeds lower at these points.

Table 59. Northbound Braley Hill Road before data.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations	44.188	7.842	38.650	4.290	37.565	3.981	38.530	3.990	277	271	268
Passenger cars	44.169	7.738	**38.692**	4.294	**37.603**	3.993	**38.572**	3.992	273	267	264
Heavy trucks	45.500	15.022	**35.750**	3.096	**35.000**	1.826	**35.750**	3.096	4	4	4
Daytime passenger cars	44.238	6.538	38.524	4.084	37.431	3.703	38.565	3.823	147	144	147
Nighttime passenger cars	44.087	8.965	38.889	4.536	37.805	4.315	38.581	4.210	126	123	117
Opposed passenger cars	43.794	7.456	38.730	4.178	36.983	4.312	38.381	4.046	63	60	63
Unopposed passenger cars	44.281	7.835	38.681	4.338	37.783	3.889	38.632	3.983	210	207	201

Bold indicates significance at 95 percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 100 shows the observed mean, 15th percentile, and 85th percentile operating speeds for passenger cars along the study section. Figure 100 also shows the mean speed for trucks within the study section. As the figure shows, the passenger car speeds and truck speeds decelerated from the control point to the approach of the curve and then stabilized from the approach to midpoint of the curve. The 85th percentile speeds for passenger cars along the curve were higher than the PSL of 40 mph. Both the passenger car and truck mean speeds along the curve were lower than the PSL of 40 mph. The mean acceleration rate from the PC to the midpoint of the curve was 0.634 ft/s for passenger cars and 0.425 ft/s for trucks.

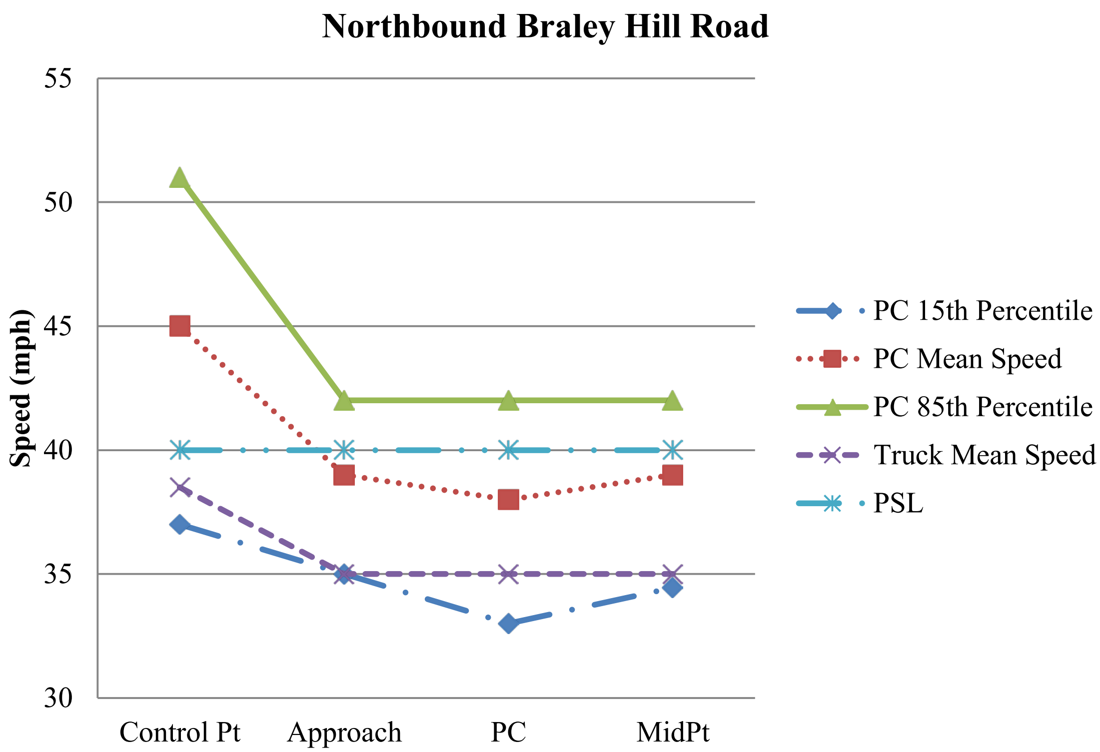


Figure 100. Graph. Graphical representation of speeds on northbound Braley Hill Road.

COMPARISON OF BEFORE AND AFTER PERIOD SPEED DATA

After applying the OSBs, the research team collected the first after period data in Arizona in November 2012 and the second after period data in April 2013. In Massachusetts, the research team collected after period data in December 2012. In Alabama, the research team collected the first after period data in October 2013 and the second after period data in December 2013. The following sections describe the results of statistical comparisons between the before and after conditions at these sites.

Before beginning the data analysis, the research team screened all raw data to exclude vehicles that could not be tracked completely from the control point to the curve midpoint. The analysis considered only free-flow vehicles. As previously defined, a vehicle was considered free-flow if it had headway of 5 s or greater. It was anticipated that vehicles present in the opposing travel lane would influence the speed of free-flow vehicles in the subject travel lane; therefore, the research team added an indicator variable to the data files to identify free-flow vehicles in the subject travel lane that were measured in the presence of an opposing vehicle. Missing data values were excluded from the analysis. Data points considered outliers were carefully evaluated to determine whether they should be eliminated or included in the analysis. Examples of outliers include vehicles traveling at very low speeds prior to entering or exiting nearby driveways.

Mean Operating Speeds

After data screening, the research team calculated mean operating speeds at each sensor location for all sites and all data-collection periods. The research team also performed an initial comparison between corresponding speed parameters at each site for each data-collection period by calculating the numerical differences in these speed parameters. The *t*-statistic for independent samples was then applied to determine if the differences in mean speeds were statistically significant. The mean operating speed comparisons were identified for each data-collection site, organized by the State.

Arizona

This section of the report presents the before-after comparisons for the four Arizona OSB sites.

Northbound Pierce Ferry Road, AZ

As table 60 shows, in the first after period at northbound Pierce Ferry Road, operating speeds remained the same for most metrics. The only metric that changed at the control point was nighttime passenger car speeds, which decreased by approximately 3.6 mph, while all other metrics did not change from the before to after period. The speeds at the other data-collection points remained the same for nighttime passenger cars. The only other significant differences between the before and first after period were daytime passenger cars and unopposed passenger cars speeds at the curve midpoint, which decreased by approximately 1.4 and 1.5 mph, respectively.

In the second after period, there were several additional significant changes between the before and second after period and between the two after periods. The only change at the control point was for opposed passenger cars, where speeds significantly increased by 3.1 mph in the second after period from the first after period. The speeds at the approach significantly increased for the following speed metrics: all observations, passenger cars, heavy trucks, nighttime passenger cars, and unopposed passenger cars, with speed increases of 1.7, 1.4, 6.8, 4.4, and 1.4 mph, respectively. Despite these significant increases at the approach, the speeds at the PC significantly decreased for all speed metrics, except for heavy trucks. These reductions ranged from approximately 4.6 to 5.6 mph. However, these reductions were not maintained throughout the curve, and speeds actually significantly increased at the curve midpoint for all speed metrics. These increases ranged from approximately 5.1 to 11.0 mph.

Overall, the OSBs significantly reduced speeds at the PC (end of the OSB), but these reductions were not observed throughout the curve, and speeds actually increased at the curve midpoint after the application of the OSBs. It is possible that drivers adjusted their speeds upward to compensate for the slower speeds on the approach to the curve.

Table 60. Before-after operating speeds at northbound Pierce Ferry Road.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations before	48.532	7.764	46.165	8.162	46.432	6.817	38.056	6.841	218	176	214
All observations 1st after	48.777	8.326	47.179	8.799	46.416	7.597	37.233	6.347	296	173	288
All observations 2nd after	49.318	7.503	**47.860**	6.765	***41.763***	4.879	***44.271***	6.737	258	253	240
Passenger cars before	48.867	7.565	46.793	7.855	46.789	6.628	38.513	6.739	203	166	199
Passenger cars 1st after	49.411	8.095	47.863	8.559	46.873	7.625	37.536	6.327	270	157	263
Passenger cars 2nd after	49.868	7.405	**48.209**	6.768	***41.969***	4.943	***44.395***	6.954	234	229	218
Heavy trucks before	44.000	9.220	37.667	7.697	40.500	7.546	32.000	5.237	15	10	15
Heavy trucks 1st after	42.192	7.965	40.077	8.236	41.938	5.802	34.040	5.755	26	16	25
Heavy trucks 2nd after	43.958	6.355	***44.458***	5.831	39.792	3.753	***43.045***	3.885	24	24	22
Daytime passenger cars before	49.577	7.116	47.521	7.429	47.669	6.277	39.271	6.744	142	118	140
Daytime passenger cars 1st after	50.353	7.825	48.263	8.303	47.519	7.133	**37.836**	6.204	232	129	225
Daytime passenger cars 2nd after	50.117	7.318	48.089	6.534	***42.133***	4.912	***44.414***	6.913	214	211	198
Nighttime passenger cars before	47.213	8.347	45.098	8.594	44.625	7.028	36.712	6.430	61	48	59
Nighttime passenger cars 1st after	**43.658**	7.390	45.421	9.747	43.893	9.138	35.763	6.832	38	28	38
Nighttime passenger cars 2nd after	*47.200*	7.997	**49.500**	9.012	***40.056***	5.047	***44.200***	7.537	20	18	20
Opposed passenger cars before	47.806	6.122	46.444	8.833	47.621	7.683	38.029	6.671	36	29	35
Opposed passenger cars 1st after	49.787	8.172	47.872	7.881	47.536	7.923	39.370	6.839	47	28	46
Opposed passenger cars 2nd after	**50.870**	7.589	48.130	7.055	***41.981***	5.443	***43.731***	6.754	54	52	52

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
Unopposed passenger cars before	49.096	7.838	46.868	7.655	46.613	6.400	38.616	6.769	167	137	164
Unopposed passenger cars 1st after	49.332	8.096	47.861	8.712	46.729	7.583	**37.147**	6.160	223	129	217
Unopposed passenger cars 2nd after	49.567	7.344	**48.233**	6.700	***41.966***	4.803	***44.602***	7.023	180	177	166

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 101 shows a graphical representation of the speed profiles for northbound Pierce Ferry Road. At the approach point and PC, the general shape of the speed profiles for the three collection periods remained relatively similar. At the midpoint, the mean and 85th percentile speeds showed an increase in the second after period.

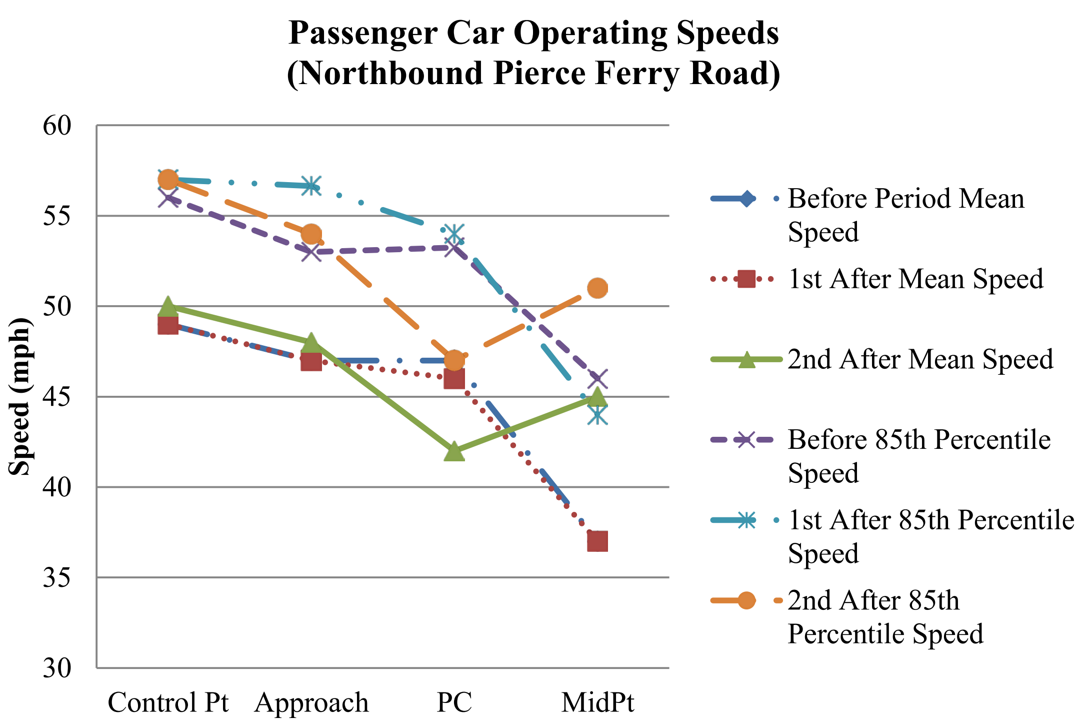


Figure 101. Graph. Operating speeds comparison on northbound Pierce Ferry Road (PSL = 55 mph).

Southbound Shinarump Road, AZ

Table 61 shows speeds increased at the control point for the following speed metrics: all observations, passenger cars, nighttime passenger cars, and unopposed passenger cars, with the increases being 1.2, 1.4, 2.2, and 1.4 mph, respectively. All observations and heavy truck speeds decreased at the control point by 1.4 and 5.8 mph, respectively. The only other change in the first after period was heavy truck speeds, which decreased at the PC by 3.8 mph and at the curve midpoint by 4.7 mph. There was no strong trend initially in speed changes after the OSBs were installed, with only heavy truck speeds decreasing at the PC and curve midpoint.

There were more changes in speeds during the second after period. Speeds remained relatively similar at the control point, with only daytime passenger car speeds increasing by approximately 1.6 mph. There were no speed changes at the approach. Speeds increased at the PC for all observations, heavy trucks, and daytime passenger cars, with speed increases of approximately 1.9, 6.1, and 2.0 mph, respectively. However, the 2.0 mph increase for daytime passenger cars was not significant after accounting for the speed increase at the control point. Speeds at the curve midpoint significantly decreased for every speed metric. The speed reductions ranged from 6.0 to 8.2 mph. Based on the second after period, speeds at the curve midpoint significantly decreased after the application of the OSBs, with a practical decrease (greater than 5 mph).

Table 61. Before-after operating speeds at southbound Shinarump Road.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations before	47.317	6.928	51.099	8.603	47.299	6.864	51.100	8.042	303	288	209
All observations 1st after	**48.542**	6.689	**49.716**	7.630	46.393	6.947	49.838	8.216	236	224	154
All observations 2nd after	47.979	8.414	***51.277***	9.756	***49.244***	9.528	***44.662***	6.987	242	176	225
Passenger cars before	47.566	6.768	51.241	8.023	48.017	6.341	51.537	7.773	249	238	175
Passenger cars 1st after	**48.930**	6.887	50.592	7.471	47.409	6.490	50.780	8.036	201	193	132
Passenger cars 2nd after	48.375	8.105	51.491	9.502	***49.158***	9.245	***45.099***	6.954	216	158	203
Heavy trucks before	46.167	7.583	50.444	10.956	43.880	8.188	48.853	9.099	54	50	34
Heavy trucks 1st after	46.314	4.928	**44.686**	6.597	**40.065**	6.413	**44.182**	7.062	35	31	22
Heavy trucks 2nd after	44.692	10.248	***49.500***	11.724	***50.000***	12.020	***40.636***	6.067	26	18	22
Daytime passenger cars before	46.686	7.001	50.777	8.431	47.009	6.033	50.894	7.375	121	115	94
Daytime	47.571	6.276	50.420	7.582	46.654	6.513	50.759	7.871	112	107	79

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
passenger cars 1st after											
Daytime passenger cars 2nd after	**48.316**	8.146	51.181	9.422	***49.017***	9.247	***44.805***	6.618	171	119	159
Nighttime passenger cars before	48.398	6.458	51.680	7.624	48.959	6.500	52.284	8.193	128	123	81
Nighttime passenger cars 1st after	**50.640**	7.266	50.809	7.365	48.349	6.374	50.811	8.353	89	86	53
Nighttime passenger cars 2nd after	48.600	8.032	52.667	9.819	49.590	9.346	***46.159***	8.046	45	39	44
Opposed passenger cars before	45.967	6.646	48.200	5.580	45.933	4.982	50.875	6.556	30	30	24
Opposed passenger cars 1st after	46.263	5.086	49.211	5.127	46.105	5.259	52.636	5.608	19	19	11
Opposed passenger cars 2nd after	47.889	7.192	49.741	5.769	48.526	7.516	***44.400***	5.276	27	19	25
Unopposed passenger cars before	47.785	6.770	51.658	8.224	48.317	6.468	51.642	7.964	219	208	151
Unopposed passenger cars 1st after	**49.209**	7.000	50.736	7.670	47.552	6.608	50.612	8.218	182	174	121
Unopposed passenger cars 2nd after	48.444	8.242	51.741	9.907	***49.245***	9.477	***45.197***	7.165	189	139	178

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 102 shows a graphical representation of the speed profiles for southbound Shinarump Road. The general shape of the mean and 85th percentile speed profiles remained relatively similar in the before and first after period. In the second after period, the mean and 85th percentile speeds increased at the midpoint.

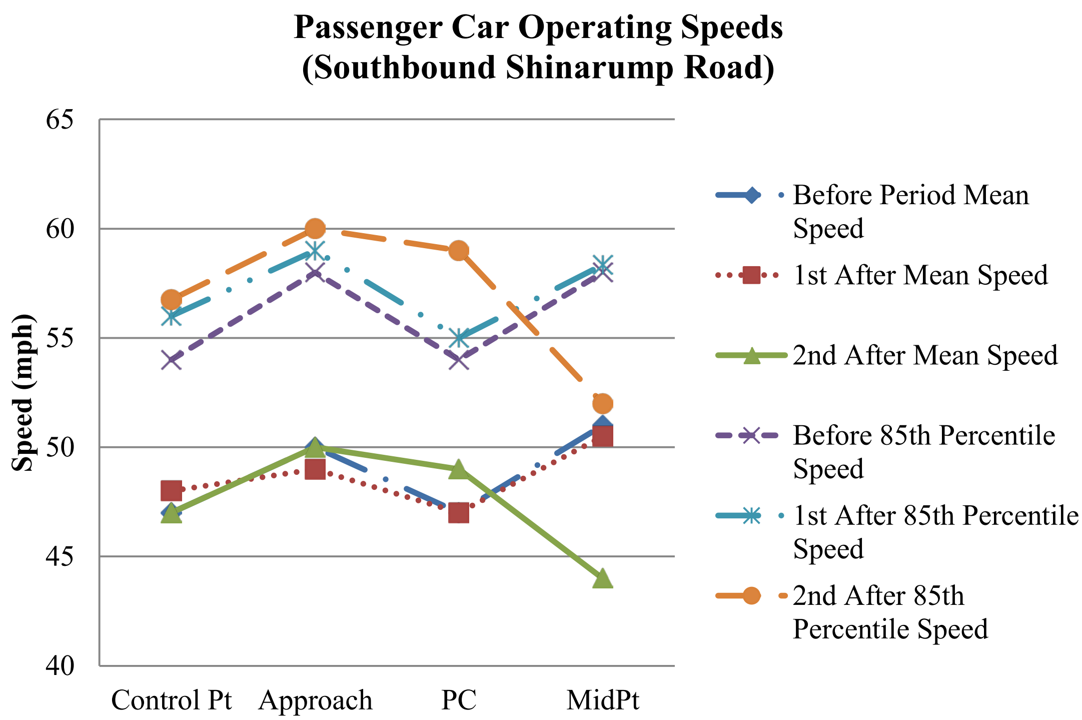


Figure 102. Graph. Operating speeds comparison on southbound Shinarump Road (PSL = 45 mph).

Southbound Diamond Bar Road, AZ

As table 62 shows, in the first after period at southbound Diamond Bar Road, operating speeds remained the same for most metrics. The only significant change at the control point was heavy truck speeds decreased by approximately 4.0 mph. Heavy truck speeds also decreased at the approach by approximately 2.7 mph, while all other speed metrics did not change significantly. The only speed metric at the PC that changed was nighttime passenger car speeds, which decreased by approximately 4.9 mph. The only speed metric at the curve midpoint that changed was heavy truck speeds, which decreased by approximately 4.7 mph. After taking into account the reduction in heavy truck speeds at the control point speed, truck speeds did not decrease in the first after period at the curve midpoint. The OSBs had virtually no effect on any of the other speed metrics in the first after period.

There were more speed changes in the second after period when compared with the before period, than there were between the first after period and before period. The only change at the control point was heavy truck speeds, which decreased from both the before period and first after period. These speed reductions were 4.0 and 6.9 mph, respectively. Speeds for all metrics increased at the approach, except heavy truck speeds did not change from the before period. Speed increases ranged between approximately 3.7 and 6.7 mph at this data-collection point. Speeds did not significantly increase at the PC, except heavy truck speeds decreased by approximately 5.2 mph. All measured speed metrics decreased at the curve midpoint, with speed decreases ranging between approximately 5.0 and 10.6 mph.

Based on the second after period, speeds at the curve midpoint significantly decreased after the application of the optical speed bars, with a practical decrease (greater than 5 mph).

Table 62. Before-after operating speeds at southbound Diamond Bar Road.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations before	53.421	8.312	49.171	6.730	50.299	7.116	50.435	7.621	292	214	209
All observations 1st after	52.111	9.266	48.144	6.865	49.969	8.313	49.091	8.654	153	64	110
All observations 2nd after	53.317	8.817	***54.410***	7.940	51.506	6.705	***42.695***	6.953	183	89	177
Passenger cars before	53.867	8.389	49.516	6.675	50.874	6.996	50.703	7.611	256	182	182
Passenger cars 1st after	53.476	9.418	49.097	7.084	50.925	8.655	50.375	8.743	124	53	88
Passenger cars 2nd after	54.071	8.616	***55.035***	7.831	52.083	6.446	***43.061***	7.017	170	84	164
Heavy trucks before	50.250	7.064	46.722	6.704	47.031	7.014	48.630	7.581	36	32	27
Heavy trucks 1st after	**46.276**	5.738	**44.069**	3.760	45.364	4.202	**43.955**	6.122	29	11	22
Heavy trucks 2nd after	***43.462***	4.502	***46.231***	3.811	***41.800***	1.483	***38.077***	3.904	13	5	13
Daytime passenger cars before	54.061	8.582	49.873	7.057	51.706	7.113	51.109	7.876	181	126	128
Daytime passenger cars 1st after	53.718	9.678	49.500	7.121	52.318	8.596	50.831	9.011	110	44	77
Daytime passenger cars 2nd after	53.832	8.857	***54.904***	8.035	52.576	6.361	***42.475***	7.134	125	66	122
Nighttime passenger cars before	53.400	7.939	48.653	5.598	49.000	6.399	49.741	6.918	75	56	54
Nighttime passenger cars 1st after	51.571	7.035	45.929	6.120	**44.111**	5.183	47.182	5.930	14	14	11
Nighttime passenger cars 2nd after	54.733	7.964	***55.400***	7.309	***50.278***	6.614	**44.762**	6.446	45	18	42
Opposed passenger cars before	53.472	9.376	50.057	6.320	51.000	8.218	50.057	8.359	53	35	35
Opposed passenger cars 1st after	52.500	10.988	48.611	7.047	52.714	9.050	52.077	9.996	18	7	13

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
Opposed passenger cars 2nd after	53.889	5.989	***53.722***	6.115	52.692	4.461	***42.588***	6.820	18	13	17
Unopposed passenger cars before	53.970	8.134	49.374	6.773	50.844	6.705	50.857	7.445	203	147	147
Unopposed passenger cars 1st after	53.642	9.175	49.179	7.120	50.652	8.665	50.080	8.548	106	46	75
Unopposed passenger cars 2nd after	54.092	8.890	***55.191***	8.012	51.972	6.765	***43.116***	7.060	152	71	147

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 103 shows a graphical representation of the speed profiles for southbound Diamond Bar Road. The general shape of the mean and 85th percentile speed profiles remained relatively similar in the before and first after period. In the second after period, the mean and 85th percentile speeds increased at the approach and midpoints.

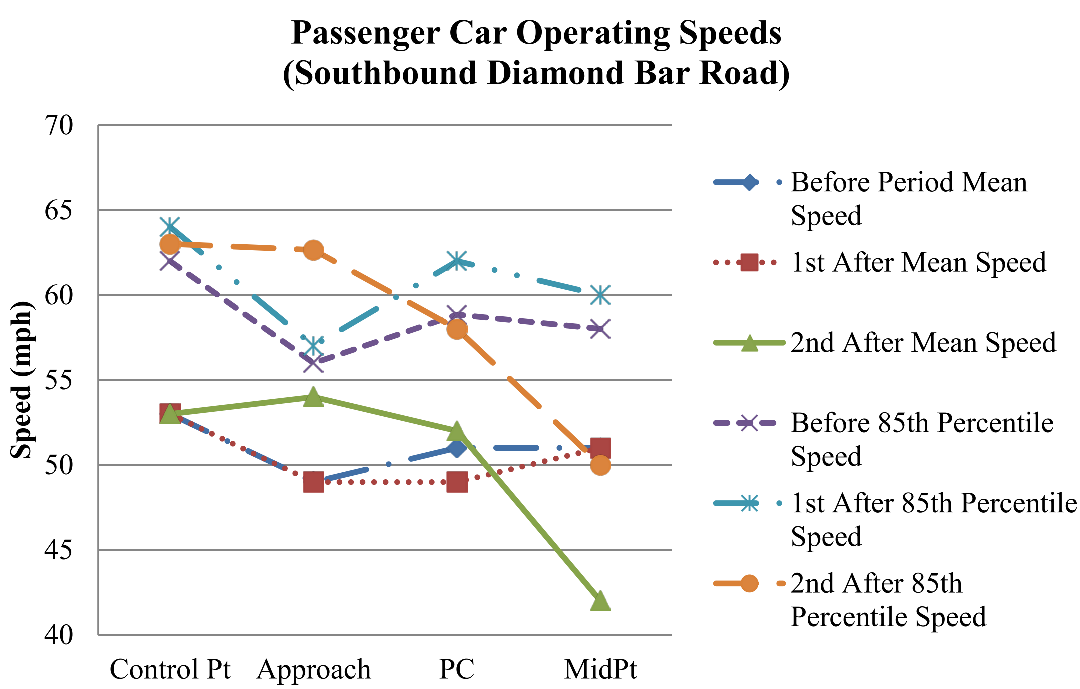


Figure 103. Graph. Operating speeds comparison on southbound Diamond Bar Road (PSL = 45 mph).

Southbound County Route 1, AZ

As table 63 shows, in the first after period at southbound County Route 1, operating speeds remained the same for most metrics. There was a significant increase at the control point for the following speed metrics: all observations, passenger cars, daytime passenger cars, nighttime passenger cars, and unopposed passenger cars, with speed increases of approximately 2.0, 2.2, 2.8, and 2.1 mph, respectively. There were no significant speed changes at the approach. Speeds at the PC remained the same in the first after period, except daytime passenger car speeds significantly decreased by approximately 1.1 mph. Speeds also remained constant at the curve midpoint, with the exception that opposed passenger car speeds significantly increased by approximately 3.4 mph. The optical speed bars had virtually no effect on speeds in the first after period.

There were more speed changes in the second after period when compared with the before period than there were between the first after period and before period. There were no significant speed differences at the control point between the second after period and before period. Speeds generally decreased from the first to second after period, with speeds returning closer to the before period. Speeds remained the same at the approach, except speeds significantly increased for all observations, passenger cars, and unopposed passenger cars, increasing by approximately 1.1, 1.0, and 1.1 mph, respectively. Speeds significantly increased at the PC for all measured speed metrics, expect opposed passenger car speeds remained the same. Speed increases ranged between approximately 1.8 and 6.3 mph. These speed increases were also observed at the curve midpoint, with all measured speed metrics, except heavy trucks, increasing significantly by approximately 3.5 to 7.1 mph.

The OSBs did not have a strong effect on operating speeds in the first after period, but after several months, the speeds at the PC and curve midpoint significantly increased between 2.1 and 7.1 mph. It is possible that drivers decelerated on the curve approach but increased speeds through the curve influenced by the large curve radii.

Table 63. Before-after operating speeds at southbound County Route 1.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations before	53.471	8.882	47.731	5.559	44.021	5.211	43.919	7.259	342	329	307
All observations 1st after	**55.448**	9.376	47.924	5.972	43.565	5.031	43.677	7.210	382	363	341
All observations 2nd after	*53.622*	9.856	***48.820***	5.650	***46.521***	5.742	***48.014***	7.115	233	219	145
Passenger cars before	53.532	8.901	47.810	5.515	44.134	5.125	43.918	7.262	327	314	292
Passenger cars 1st after	**55.746**	9.419	48.045	5.896	43.635	5.013	43.877	7.147	355	337	316
Passenger cars 2nd after	***53.532***	9.792	**48.792**	5.656	***46.417***	5.719	***47.986***	7.166	216	204	138
Heavy trucks before	52.133	8.651	46.000	6.414	41.667	6.532	43.933	7.450	15	15	15
Heavy trucks 1st after	51.519	7.939	46.333	6.811	42.654	5.284	41.160	7.679	27	26	25

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
Heavy trucks 2nd after	54.765	10.889	49.176	5.736	***47.933***	6.076	*48.571*	6.477	17	15	7
Daytime passenger cars before	53.847	9.266	49.038	5.430	44.977	4.895	44.599	7.081	183	174	162
Daytime passenger cars 1st after	**55.689**	9.357	48.222	5.420	**43.895**	4.746	43.969	7.107	257	248	229
Daytime passenger cars 2nd after	***54.046***	9.582	***49.408***	5.225	***47.034***	5.326	***48.316***	7.149	152	145	98
Nighttime passenger cars before	53.132	8.428	46.250	5.238	43.086	5.228	43.069	7.421	144	140	130
Nighttime passenger cars 1st after	**55.898**	9.628	47.582	7.004	42.910	5.656	43.632	7.287	98	89	87
Nighttime passenger cars 2nd after	***52.313***	10.249	47.328	6.375	***44.898***	6.383	***47.175***	7.236	64	59	40
Opposed passenger cars before	53.479	9.299	47.125	5.354	44.111	4.900	41.091	7.844	48	45	44
Opposed passenger cars 1st after	56.000	9.372	48.301	5.721	43.843	5.029	**44.506**	6.613	93	89	85
Opposed passenger cars 2nd after	53.407	9.616	47.963	4.642	45.373	5.564	***48.171***	6.667	54	51	41
Unopposed passenger cars before	53.541	8.848	47.928	5.543	44.138	5.171	44.419	7.053	279	269	248
Unopposed passenger cars 1st after	**55.656**	9.452	47.954	5.965	43.560	5.015	43.645	7.333	262	248	231
Unopposed passenger cars 2nd after	***53.574***	9.879	***49.068***	5.943	***46.765***	5.745	***47.907***	7.399	162	153	97

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 104 shows a graphical representation of the speed profiles for southbound County Route 1. The general shape of the mean and 85th percentile speed profiles, at the curve approach point and PC, for the three collection periods remained relatively similar in the before and first after period. However, the midpoint speeds increased in the second after period.

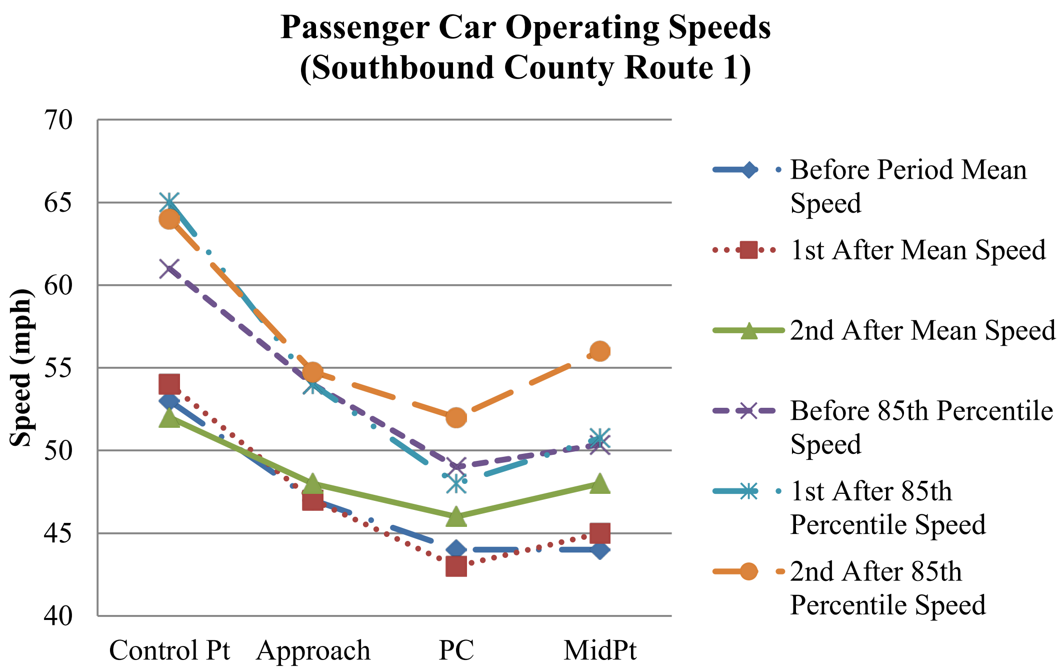


Figure 104. Graph. Operating speeds comparison on southbound County Route 1 (PSL = 35 mph).

Arizona Summary of Results

Overall, the OSBs did not have an effect on operating speeds at the Arizona sites. There were no changes in speeds that were consistent across all of the sites.

Alabama

This section of the report presents the before-after comparisons for the eight Alabama OSB sites.

Alabama Location #1

As table 64 shows, in the first after period at Alabama Location #1, operating speeds decreased for several speed metrics. There was a significant decrease at the control point for all speed metrics. These speed decreases ranged between approximately 2.1 and 6.7 mph. Speeds at the approach significantly decreased for all speed metrics. However, after accounting for the speed decrease at the control point, none of these reductions were significant, and speeds for passenger cars, daytime passenger cars, and unopposed passenger cars actually increased. Speeds at the PC significantly decreased for the following speed metrics: all observations, passenger cars, daytime passenger cars, and unopposed passenger cars. However, after accounting for the speed decrease at the control point, none of these reductions were significant, and speeds for passenger cars and opposed passenger cars actually increased. Speeds at the curve midpoint did not change significantly from the before period. If the speed reduction at the control point was taken into account, speeds increased slightly at the curve midpoint.

There were more speed changes in the second after period when compared with the before period, than there were between the first after period and before period. Speeds at the control point increased for all speed metrics, except for heavy trucks. The range of speed increases was between approximately 1.7 and 2.7 mph. Not taking the speed increase at the control point into account, speeds at the approach increased from the first after period, but they remained the same when compared with the before period, except all observations significantly decreased by approximately 0.8 mph. Not taking into account the speed increase at the control point, speeds at the PC increased from the first after period, but remained the same when compared with the before period, except opposed passenger cars significantly increased by approximately 3.3 mph. Speeds significantly decreased at the curve midpoint for all speed metrics, except for heavy trucks. The speed decreases were between approximately 1.4 and 2.6 mph. If the speed increases at the control point were taken into account, these decreases became more significant.

The OSBs did not have a strong effect on operating speeds in the first after period, but after several months, the speeds at the curve midpoint significantly decreased between 1.4 and 2.6 mph.

Table 64. Before-after operating speeds at Alabama Location #1.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations before	47.883	9.368	49.208	7.596	50.103	8.770	42.323	6.912	375	87	353
All observations 1st after	**43.306**	6.928	**45.497**	6.752	**47.431**	5.886	41.691	6.951	340	204	333
All observations 2nd after	***50.190***	7.863	***48.380***	6.788	***49.776***	6.830	***40.364***	9.437	822	223	733
Passenger cars before	47.917	9.380	49.218	7.540	50.429	8.700	42.516	6.892	362	84	341
Passenger cars 1st after	**43.575**	6.880	**46.134**	6.490	**48.148**	5.530	42.523	6.342	313	183	306
Passenger cars 2nd after	***50.226***	7.931	***48.522***	6.792	***49.906***	6.914	***40.475***	9.413	793	213	710
Heavy trucks before	46.923	9.332	48.923	9.394	41.000	6.083	36.833	5.132	13	3	12
Heavy trucks 1st after	**40.185**	6.850	**38.111**	5.243	41.190	5.269	**32.259**	6.671	27	21	27
Heavy trucks 2nd after	***49.207***	5.722	***44.483***	5.468	***47.000***	3.944	***36.957***	9.740	29	10	23
Daytime passenger cars before	47.638	9.418	48.805	7.641	50.795	8.551	42.288	6.764	174	39	160
Daytime passenger cars 1st after	**42.964**	6.865	**46.016**	6.568	**48.020**	5.060	42.470	6.112	252	151	247
Daytime passenger cars 2nd after	***49.417***	7.985	***48.296***	6.904	***49.578***	7.058	***40.840***	9.469	348	102	306
Nighttime passenger cars before	48.176	9.362	49.601	7.444	50.111	8.912	42.718	7.016	188	45	181

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
Nighttime passenger cars 1st after	**46.098**	6.395	**46.623**	6.186	48.750	7.427	42.746	7.282	61	32	59
Nighttime passenger cars 2nd after	***50.858***	7.838	***48.699***	6.705	50.207	6.797	***40.198***	9.373	445	111	404
Opposed passenger cars before	47.529	10.661	48.793	7.385	46.833	7.648	42.037	6.512	87	24	82
Opposed passenger cars 1st after	**43.250**	5.583	**44.775**	4.699	46.667	4.151	41.725	6.345	40	21	40
Opposed passenger cars 2nd after	***49.228***	7.986	***47.772***	6.816	***50.130***	5.361	***39.479***	10.074	158	46	142
Unopposed passenger cars before	48.040	8.955	49.353	7.596	51.867	8.736	42.668	7.014	275	60	259
Unopposed passenger cars 1st after	**43.623**	7.057	**46.333**	6.696	**48.340**	5.666	42.643	6.345	273	162	266
Unopposed passenger cars 2nd after	***50.474***	7.904	***48.709***	6.778	***49.844***	7.297	***40.724***	9.233	635	167	568

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at 95 percent confidence level (α=0.05) between the 1st and 2nd after periods
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 105 shows a graphical representation of the speed profiles for Alabama Location #1. The mean and 85th percentile operating speeds increased in the first after period but remained relatively similar during the before and second after data-collection periods.

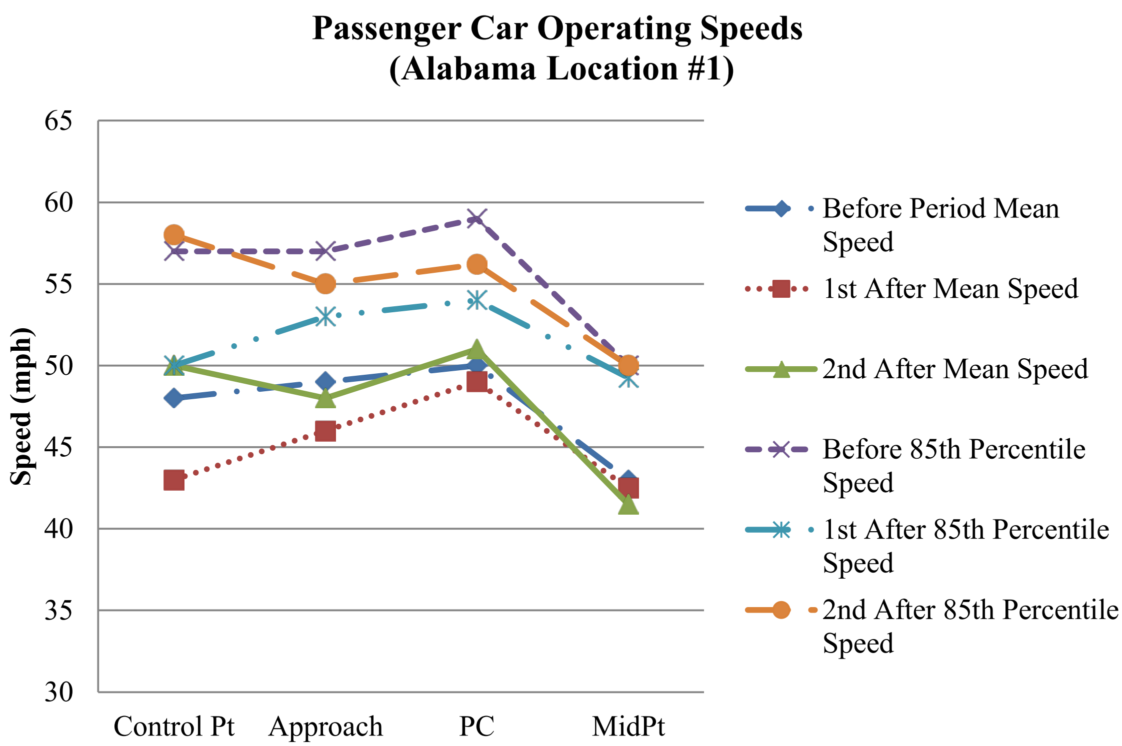


Figure 105. Graph. Operating speeds comparison at Alabama Location #1 (PSL = 55 mph).

Alabama Location #2

As table 65 shows, in the first after period at Alabama Location #2, operating speeds remained the same for most metrics. The only significant change at the control point was unopposed passenger car speeds decreased by approximately 1.1 mph. Speeds at the approach increased for the following speed metrics: all observations, passenger cars, and opposed passenger cars, with speed increases of 1.0, 1.1, and 2.4 mph, respectively. There were no significant speed changes at the PC. Speeds at the curve midpoint significantly increased for the following speed metrics: all observations, passenger cars, daytime passenger cars, and opposed passenger cars, with speed increases of approximately 1.0, 1.1, 1.9, and 3.4 mph, respectively. After installing the OSBs, speeds significantly increased at the curve midpoint during the first after period, but most of the speed increases were not practically significant.

The speed changes that occurred in the first after period continued through the second after period, and speeds in the second after period were similar to the first after period. The only significant change at the control point was opposed passenger car speeds increased by approximately 2.1 mph. Speeds at the approach increased for the same speed metrics as in the first after period: all observations, passenger cars, and opposed passenger cars, with speed increases of approximately 1.0, 0.9, and 3.5 mph, respectively. There were no significant speed changes at the PC. Speeds increased at the curve midpoint for the following speed metrics: all observations, daytime passenger cars, and opposed passenger cars, with speed increases of approximately 5.6, 1.5, and 3.0 mph, respectively. However, if the speed increase at the control point was taken into account, the speed increase at the curve midpoint for opposed passenger cars was no longer significant. Speeds significantly increased for several speed metrics in both after periods at the curve midpoint, but many speed metrics did not increase.

Table 65. Before-after operating speeds at Alabama Location #2.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations before	49.269	7.604	44.291	5.822	42.472	5.747	38.363	7.094	398	290	358
All observations 1st after	48.843	6.991	**45.328**	5.034	42.245	4.882	**39.310**	5.429	470	413	462
All observations 2nd after	49.374	6.823	**45.335**	5.407	***43.163***	5.419	***43.949***	6.719	495	307	316
Passenger cars before	49.569	7.238	44.479	5.623	42.630	5.554	38.418	7.129	376	273	336
Passenger cars 1st after	49.039	6.984	**45.549**	4.977	42.552	4.742	**39.556**	5.325	439	386	432
Passenger cars 2nd after	49.472	6.787	**45.413**	5.181	***43.192***	5.263	39.003	5.624	632	499	585
Heavy trucks before	44.136	11.319	41.091	8.053	39.941	8.050	37.091	6.560	22	17	22
Heavy trucks 1st after	46.065	6.582	42.194	4.861	37.852	4.825	35.767	5.764	31	27	30
Heavy trucks 2nd after	46.429	7.386	43.000	10.015	***42.200***	9.466	37.111	8.345	21	15	18
Daytime passenger cars before	49.884	8.070	45.072	5.722	43.263	5.737	37.821	7.802	138	99	122
Daytime passenger cars 1st after	49.302	6.916	45.764	4.782	42.782	4.626	**39.770**	5.158	318	280	313
Daytime passenger cars 2nd after	49.584	6.915	45.776	5.031	***43.692***	5.102	**39.381**	5.647	375	295	344
Nighttime passenger cars before	49.387	6.721	44.134	5.548	42.270	5.432	38.762	6.720	238	174	214
Nighttime passenger cars 1st after	48.347	7.143	44.983	5.437	41.943	5.008	38.992	5.725	121	106	119
Nighttime passenger cars 2nd	49.307	6.607	44.883	5.359	42.471	5.419	38.465	5.557	257	204	241

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
after											
Opposed passenger cars before	47.809	7.949	43.022	6.312	41.721	5.057	36.378	7.241	89	61	82
Opposed passenger cars 1st after	49.026	6.713	**45.421**	4.674	42.338	5.027	**39.787**	5.602	76	68	75
Opposed passenger cars 2nd after	**49.918**	6.386	**46.541**	4.829	***44.012***	4.633	**39.337**	5.721	98	81	92
Unopposed passenger cars before	50.115	6.928	44.930	5.323	42.892	5.674	39.075	6.976	287	212	254
Unopposed passenger cars 1st after	**49.041**	7.048	45.576	5.044	42.597	4.686	39.507	5.272	363	318	357
Unopposed passenger cars 2nd after	49.390	6.861	45.206	5.221	43.033	5.367	38.941	5.609	534	418	493

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 106 shows a graphical representation of the speed profiles for Alabama Location #2. The general shape of the speed profiles for the three collection periods remained relatively similar.

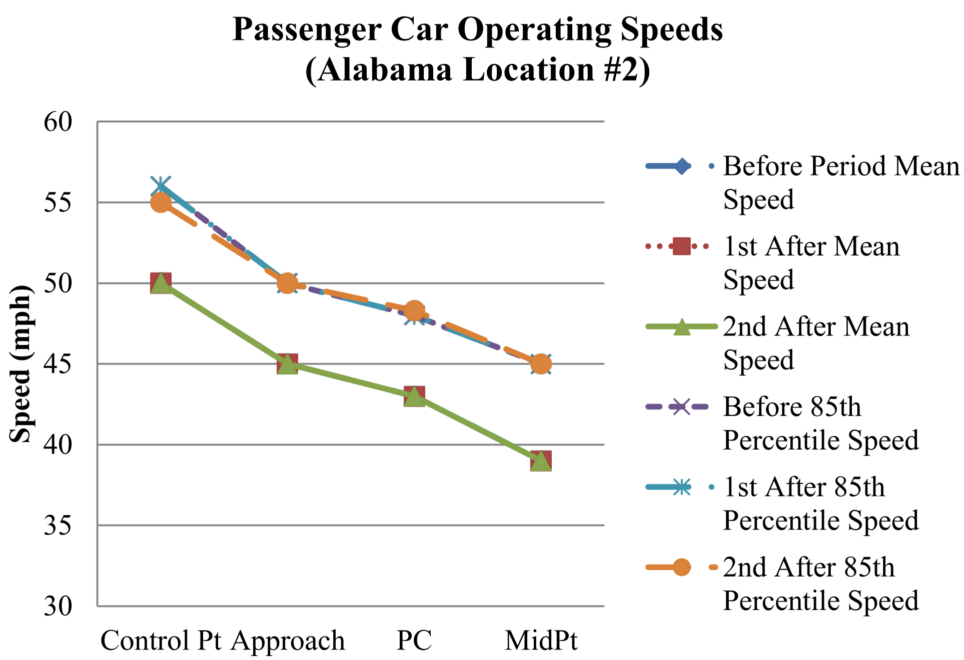


Figure 106. Graph. Operating speeds comparison at Alabama Location #2 (PSL = 55 mph).

Alabama Location #3

As table 66 shows, in the first after period at Alabama Location #3, operating speeds changed for many speed metrics. As in the before period, operating speeds at the control point were high. It was originally thought that a sensor malfunction caused these high speed measurements, but these high speeds persisted through all three time periods. Speeds at the control point increased for all speed metrics, except nighttime passenger cars. These speed increases were between 2.5 and 3.4 mph. Speeds at the approach decreased for all speed metrics, except nighttime passenger cars and opposed passenger cars. These speed decreases were between approximately 1.0 and 3.0 mph. The only change at the PC was passenger car speeds, which increased by approximately 0.6 mph. Speeds at the curve midpoint decreased for all speed metrics, except heavy trucks. These speed reductions were between approximately 1.9 and 2.8 mph. Speeds at the curve midpoint were slightly lower immediately after the OSBs were installed.

Most of the changes in speeds from the first after period did not persist through the second after period. Speeds at the control point increased for all speed metrics, with speed increases between approximately 2.0 and 3.5 mph. Speeds at the approach remained the same, except all observations decreased by approximately 0.6 mph. There were no significant speed changes at the PC. Speeds at the curve midpoint increased from the first after period and returned closer to the before period. There were no significant speed changes at the curve midpoint.

Speeds slightly decreased at the curve midpoint immediately after installing the OSBs, but the decrease was not maintained through the second after period.

Table 66. Before-after operating speeds at Alabama Location #3.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations before	65.253	12.908	49.831	6.463	41.307	4.663	41.233	5.914	498	472	472
All observations 1st after	**67.813**	11.357	**48.246**	5.062	41.365	4.737	**38.677**	6.919	513	493	511
All observations 2nd after	**68.027**	11.822	***49.208***	5.335	41.622	4.770	***41.085***	6.888	960	891	879
Passenger cars before	65.520	13.004	50.000	6.522	41.629	4.652	41.740	5.683	440	415	415
Passenger cars 1st after	**68.215**	11.509	**49.018**	4.728	**42.239**	4.371	**39.085**	7.219	400	381	398
Passenger cars 2nd after	**68.302**	11.785	***49.606***	5.297	42.065	4.712	***41.571***	6.823	819	752	748
Heavy trucks before	63.224	12.070	48.552	5.891	38.965	4.066	37.544	6.299	58	57	57
Heavy trucks 1st after	**66.389**	10.732	**45.513**	5.276	38.393	4.748	37.239	5.534	113	112	113
Heavy trucks 2nd after	**66.433**	11.955	***46.901***	4.969	39.223	4.366	38.313	6.618	141	139	131
Daytime passenger cars before	66.424	12.482	50.549	6.973	41.923	4.682	41.689	5.979	264	246	251
Daytime passenger cars 1st after	**68.880**	11.622	**49.098**	4.733	42.278	4.364	**38.861**	7.470	317	302	316
Daytime passenger cars 2nd after	**69.931**	11.578	***50.006***	5.282	42.450	4.634	***41.924***	6.831	466	436	432
Nighttime passenger cars before	64.165	13.675	49.176	5.702	41.201	4.589	41.817	5.214	176	169	164
Nighttime passenger cars 1st after	65.675	10.759	48.711	4.725	42.089	4.421	**39.951**	6.118	83	79	82
Nighttime passenger cars 2nd after	**66.150**	11.726	49.076	5.278	41.535	4.774	41.089	6.793	353	316	316
Opposed passenger cars before	65.516	13.031	49.979	6.684	41.423	5.063	41.928	6.036	188	175	180
Opposed passenger cars 1st after	**68.953**	11.438	48.929	5.002	41.900	4.482	**39.318**	6.634	85	80	85
Opposed passenger cars 2nd after	**68.935**	12.198	49.245	5.109	41.894	4.961	***42.335***	7.003	200	180	185

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
Unopposed passenger cars before	65.524	13.010	50.016	6.413	41.779	4.332	41.596	5.405	252	240	235
Unopposed passenger cars 1st after	**68.016**	11.538	**49.041**	4.659	42.329	4.344	**39.022**	7.378	315	301	313
Unopposed passenger cars 2nd after	**68.097**	11.651	***49.722***	5.355	42.119	4.634	***41.320***	6.751	619	572	563

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 107 shows a graphical representation of the speed profiles for Alabama Location #3. The general shape of the speed profiles for the three collection periods remained relatively similar.

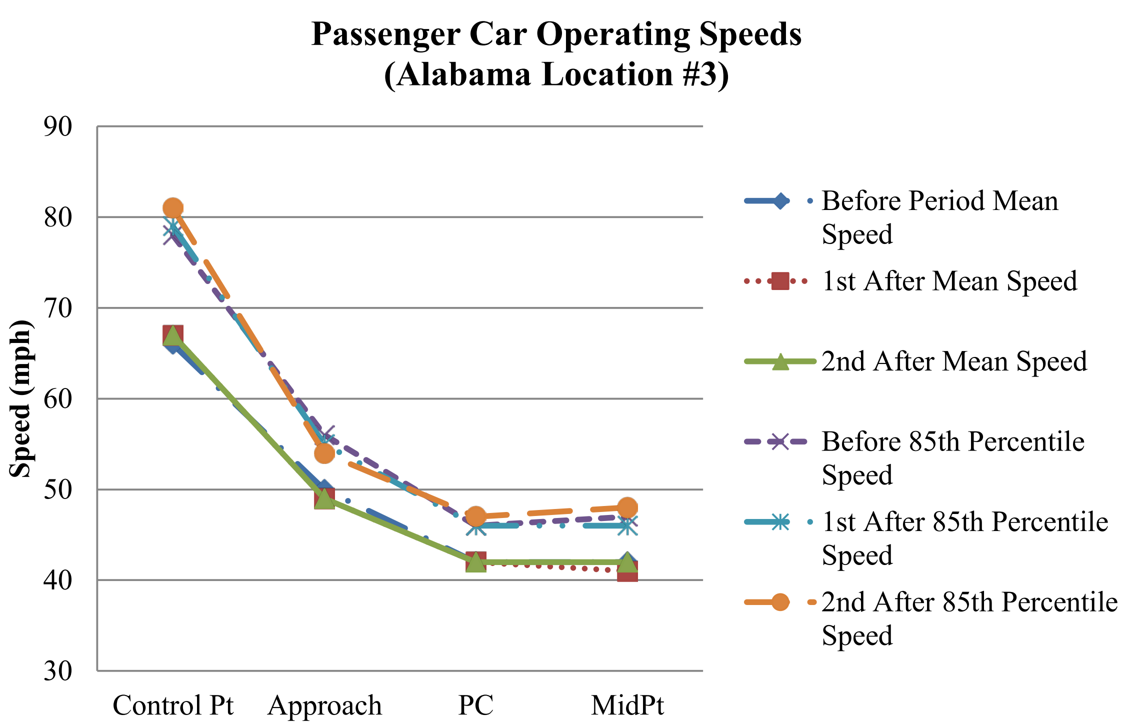


Figure 107. Graph. Operating speeds comparison at Alabama Location #3 (PSL = 55 mph).

Alabama Location #4

As Table 67 shows, in the first after period at Alabama Location #4, operating speeds decreased for many speed metrics. There was a significant decrease at the control point for all measured speed metrics. Speed decreases were between approximately 1.6 and 3.3 mph for passenger cars and 5.3 mph for heavy trucks. There was a significant decrease at the approach for all speed metrics, except heavy trucks and opposed passenger cars. These speed decreases were all approximately 1.5 mph. However, if the decrease in speed at the control point was taken into account, the speed decreases at the approach were no longer significant. There was a significant decrease at the PC for all measured speed metrics, except heavy trucks. Speed decreases were between 2.0 and 2.5 mph. However, if the speed decreases at the control point were taken into account, the only speed decrease at the PC that was significant was for daytime passenger cars, and nighttime speeds actually increase slightly. Speeds were also significantly lower at the curve midpoint for all measured speed metrics. However, if the speed decreases at the control point were taken into account, none of the speed decreases at the curve midpoint were significant. Overall, installing the OSBs did not have an effect on operating speeds in the first after period.

Most of the speed changes in the first after period were no longer present in the second after period. Speeds at the control point were closer to speeds in the before period, and there were no significant speed changes. Speeds at the approach were also closer to speeds in the before period, and there were no significant speed changes, except daytime passenger speeds increased by approximately 1.1 mph. Speeds at the PC significantly decreased for the following speed metrics: all observations, passenger cars, and opposed passenger cars, with speed decreases of approximately 0.6, 0.7, and 1.2 mph, respectively. Speeds at the curve midpoint did not significantly change, except nighttime speeds were approximately 1.5 mph lower.

Speeds slightly decreased at the curve midpoint after installing the OSBs, but the decrease was not maintained through the second after period.

Table 67. Before-after operating speeds at Alabama Location #4.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N1	N2	N3
All observations before	50.761	9.466	46.666	7.082	39.091	4.939	34.664	8.205	494	471	455
All observations 1st after	**48.515**	8.022	**45.152**	5.409	**36.891**	4.357	**31.938**	6.086	513	512	513
All observations 2nd after	***50.322***	7.655	***47.172***	6.128	***38.469***	4.754	***34.433***	8.094	615	603	564
Passenger cars before	50.633	9.369	46.703	7.019	39.153	4.883	34.506	8.197	472	452	437
Passenger cars 1st after	**48.541**	7.986	**45.242**	5.431	**37.000**	4.330	**31.819**	6.162	475	474	475
Passenger cars 2nd after	***50.381***	7.664	***47.155***	6.038	***38.455***	4.709	***34.434***	8.125	582	571	535
Heavy trucks before	53.500	11.237	45.864	8.481	37.632	6.103	38.500	7.649	22	19	18
Heavy trucks 1st after	**48.184**	8.561	44.026	5.059	35.526	4.519	**33.421**	4.864	38	38	38
Heavy trucks 2nd after	49.273	7.543	***47.485***	7.649	***38.719***	5.566	**34.414**	7.623	33	32	29
Daytime passenger cars before	50.588	9.045	46.631	7.464	39.528	5.023	33.861	8.062	187	176	173
Daytime passenger cars 1st after	**48.966**	8.241	**45.564**	5.389	**37.020**	4.325	**31.969**	5.975	351	350	351
Daytime passenger cars 2nd after	***50.324***	7.331	***47.770***	6.113	***38.953***	4.793	***35.084***	8.230	343	338	320
Nighttime passenger cars before	50.663	9.592	46.751	6.723	38.913	4.786	34.928	8.273	285	276	264
Nighttime passenger cars 1st after	**47.339**	7.111	**44.331**	5.466	**36.944**	4.362	**31.395**	6.668	124	124	124
Nighttime passenger cars 2nd after	***50.464***	8.132	***46.272***	5.829	**37.734**	4.499	***33.465***	7.887	239	233	215
Opposed passenger cars before	50.886	9.349	46.429	8.484	39.141	5.234	34.328	8.582	140	128	125

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N1	N2	N3
Opposed passenger cars 1st after	49.156	8.039	45.227	5.317	**36.836**	4.814	**31.953**	6.551	128	128	128
Opposed passenger cars 2nd after	***51.089***	7.337	***47.274***	6.375	***37.910***	4.802	***34.535***	8.608	124	122	114
Unopposed passenger cars before	50.527	9.390	46.819	6.311	39.157	4.746	34.577	8.051	332	324	312
Unopposed passenger cars 1st after	**48.314**	7.966	**45.248**	5.480	**37.061**	4.143	**31.769**	6.021	347	346	347
Unopposed passenger cars 2nd after	***50.190***	7.746	***47.122***	5.950	***38.604***	4.678	***34.406***	8.000	458	449	421

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 108 shows a graphical representation of the speed profiles for Alabama Location #4. The general shape of the speed profiles for the three collection periods remained relatively similar.

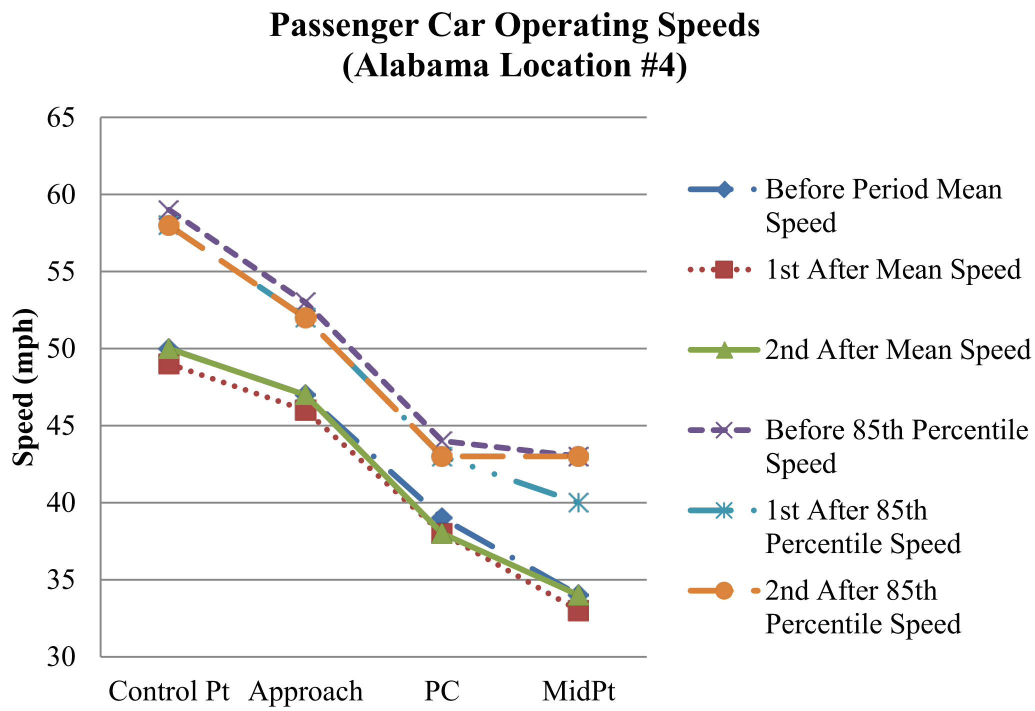


Figure 108. Graph. Operating speeds comparison at Alabama Location #4 (PSL = 35 mph).

Alabama Location #5

As table 68 shows, in the first after period at Alabama Location #5, operating speeds changed for several speed metrics. There were no significant speed changes at the control point or approach. Speeds at the PC significantly decreased for the following speed metrics: all observations, passenger cars, and unopposed passenger cars, with speed decreases of approximately 2.2, 2.1, and 2.9 mph, respectively. Speeds for opposed passenger cars significantly increased by approximately 5.4 mph. Speeds at the curve midpoint significantly increased for all speed metrics, except heavy trucks. These speed increases were between approximately 4.2 and 7.8 mph. Speeds at the PC significantly decreased after the installation of the OSBs, but vehicles maintained a more constant speed throughout the curve, and speeds increased at the curve midpoint.

The speed changes that occurred in the first after period continued through the second after period, and speeds in the second after period were similar to the first after period. There were no significant speed changes at the control point or approach. Speeds at the PC significantly decreased for the following speed metrics: all observations, passenger cars, heavy trucks, and unopposed passenger cars, with speed decreases of approximately 1.9, 1.8, 5.2, 1.7, and 2.6 mph. Speeds for opposed passenger cars significantly increased by approximately 6.5 mph. Speeds at the curve midpoint significantly increased for all speed metrics. Speed increases were between approximately 3.6 and 8.5 mph for passenger cars and 7.8 mph for heavy trucks.

Installing the OSBs decreased operating speeds at the PC during both after periods between approximately 1.7 and 2.9 mph for passenger cars, while speeds at the curve midpoint increased during both after periods between approximately 3.6 and 8.5 mph.

Table 68. Before-after operating speeds at Alabama Location #5.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N1	N2	N3
All observations before	41.944	7.212	43.742	8.327	45.781	7.524	39.810	6.842	233	64	221
All observations 1st after	41.467	7.184	43.170	7.194	**43.636**	6.437	**44.182**	7.264	353	236	242
All observations 2nd after	41.889	7.277	43.069	7.426	**43.847**	6.047	**43.949**	6.719	495	307	316
Passenger cars before	42.299	7.094	44.054	8.250	45.794	7.584	40.028	6.782	224	63	212
Passenger cars 1st after	41.851	7.086	43.460	7.037	**43.689**	6.435	**44.384**	7.164	335	222	232
Passenger cars 2nd after	42.185	7.110	43.344	7.168	**44.024**	5.960	**44.020**	6.672	471	294	302
Heavy trucks before	33.111	3.822	36.000	6.614	45.000	0.000	34.667	6.557	9	1	9
Heavy trucks 1st after	34.333	5.018	37.778	8.128	42.786	6.647	39.500	8.383	18	14	10
Heavy trucks 2nd after	36.083	8.214	37.667	10.120	**39.846**	6.842	**42.429**	7.773	24	13	14
Daytime passenger cars before	41.947	6.887	44.405	7.941	45.619	7.450	39.787	6.772	131	42	122
Daytime passenger cars 1st after	41.644	6.846	43.222	7.071	43.668	6.212	**44.273**	7.151	284	193	198
Daytime passenger cars 2nd after	42.272	6.988	43.276	7.003	43.921	6.131	**43.858**	7.030	301	190	204
Nighttime passenger cars before	42.796	7.383	43.559	8.686	46.143	8.021	40.356	6.821	93	21	90
Nighttime passenger cars 1st after	43.000	8.280	44.784	6.763	43.828	7.888	**45.029**	7.309	51	29	34
Nighttime passenger cars 2nd after	42.029	7.340	43.465	7.471	44.212	5.660	**44.357**	5.877	170	104	98

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N1	N2	N3
Opposed passenger cars before	40.235	7.242	41.353	11.045	38.833	7.333	34.933	7.045	17	6	15
Opposed passenger cars 1st after	41.905	9.286	42.762	8.630	**44.200**	6.609	**42.692**	6.851	42	30	26
Opposed passenger cars 2nd after	41.867	10.281	44.567	8.063	**45.294**	5.828	**43.389**	5.782	30	17	18
Unopposed passenger cars before	42.469	7.072	44.275	7.972	46.526	7.290	40.416	6.621	207	57	197
Unopposed passenger cars 1st after	41.843	6.733	43.560	6.790	**43.609**	6.421	**44.597**	7.190	293	192	206
Unopposed passenger cars 2nd after	42.206	6.858	43.261	7.106	**43.946**	5.970	**44.060**	6.732	441	277	284

Bold indicates significance at 95 percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at 95 percent confidence level (α=0.05) between the 1st and 2nd after periods
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 109 shows a graphical representation of the speed profiles for Alabama Location #5. The PC mean operating speeds decreased in the two after periods while the midpoint speeds increased. The midpoint 85th percentile speeds increased in the two after periods.

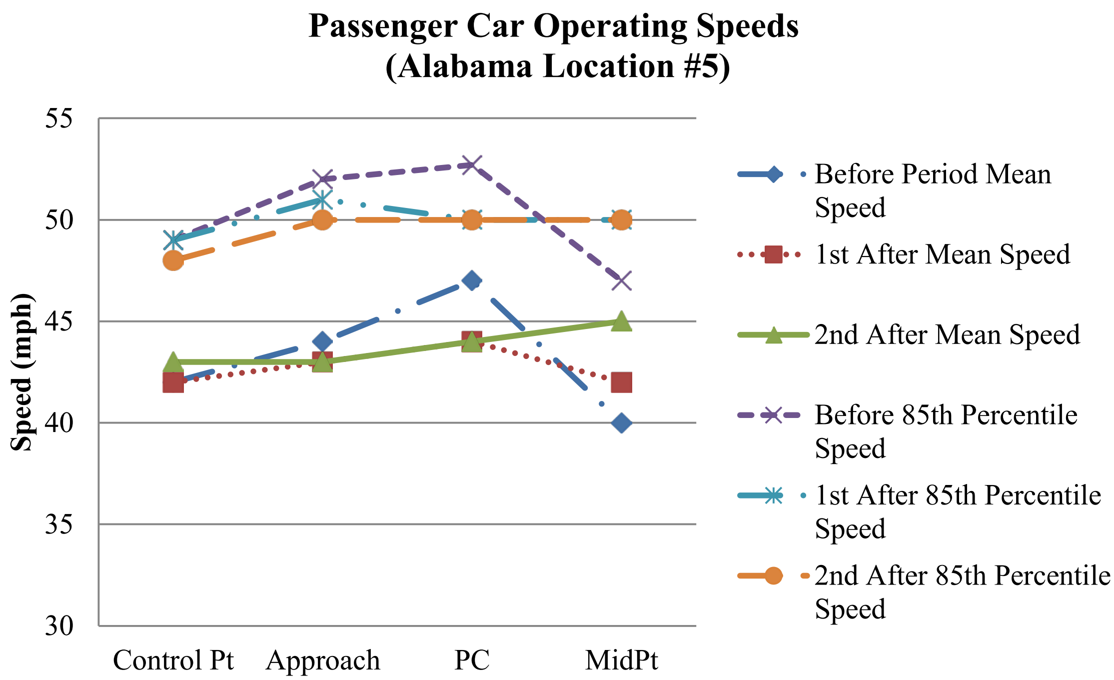


Figure 109. Graph. Operating speeds comparison at Alabama Location #5 (PSL = 40 mph).

Alabama Location #6

As table 69 shows, in the first after period at Alabama Location #6, operating speeds decreased for many speed metrics. There was a significant decrease at the control point for the following speed metrics: all observations, passenger cars, nighttime passenger cars, and unopposed passenger cars, with speed decreases of approximately 2.0, 2.0, 4.4, and 2.2 mph, respectively. There was a significant increase at the approach for all speed metrics. These increases ranged from approximately 8.3 to 9.5 mph for passenger cars. However, these increased speeds were not maintained, and speeds at the PC significantly decreased for the following speed metrics: all observations, passenger cars, heavy trucks, and unopposed passenger cars, with speed decreases of approximately 1.6, 1.4, 4.4, and 1.8 mph, respectively. Speeds were also significantly lower at the curve midpoint for all passenger car speed metrics. These speed reductions were approximately 3.9 to 8.8 mph. Speeds throughout the curve decreased between 1.4 and 8.8 mph at this site after installing the OSBs.

In the second after period, there were several additional significant changes between the before and second after period and between the two after periods. Speeds generally remained the same at the control point. Only all observations and heavy trucks had a significant reduction in speeds, with speed reductions of approximately 1.3 and 7.0 mph, respectively. Speeds at the approach were lower than in the first after period, but they were still significantly higher in the second after period for the following speed metrics: all observations, passenger cars, nighttime passenger cars, and opposed passenger cars, with speed increases of approximately 1.3, 1.2, 1.9, and 4.3 mph, respectively. Despite these significant increases at the approach, the speeds at the PC remained constant from the before to second after period with the exception of nighttime passenger cars. There was a significant increase of approximately 2.9 mph; however, after accounting for the speed increase at the approach, this increase was no longer significant.

Speeds increased slightly at the curve midpoint in the second after period when compared with the first after period for several speed metrics, but speeds were lower than the before period for all speed metrics. These speed reductions were approximately 3.2 to 4.75 mph.

Overall, the OSBs significantly reduced speeds at the curve midpoint in both the first and second after periods.

Table 69. Before-after operating speeds at Alabama Location #6.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N1	N2	N3
All observations before	43.413	9.211	45.102	7.201	46.442	6.431	45.495	7.111	196	86	97
All observations 1st after	**41.382**	9.064	**54.627**	8.674	**44.879**	6.628	**40.780**	7.109	220	190	205
All observations 2nd after	**42.082**	10.108	***46.412***	7.360	***47.644***	7.706	**41.762**	7.679	478	104	433
Passenger cars before	43.399	9.294	45.426	6.939	46.447	6.469	45.613	7.199	188	85	93
Passenger cars 1st after	**41.371**	9.028	**54.709**	8.760	**45.005**	6.642	**40.848**	7.159	213	183	198
Passenger cars 2nd after	42.326	10.069	***46.652***	7.339	***47.859***	7.775	***41.925***	7.674	457	99	415
Heavy trucks before	43.750	7.498	37.500	9.457	46.000	0.000	42.750	4.272	8	1	4
Heavy trucks 1st after	41.714	10.889	**52.143**	5.242	**41.571**	5.682	38.857	5.581	7	7	7
Heavy trucks 2nd after	**36.762**	9.705	***41.190***	5.828	43.400	4.930	**38.000**	6.979	21	5	18
Daytime passenger cars before	43.504	9.135	45.434	6.731	46.586	6.489	45.032	7.759	129	58	62
Daytime passenger cars 1st after	41.937	8.633	**54.931**	8.493	45.219	6.581	**40.810**	7.160	175	151	163
Daytime passenger cars 2nd after	42.604	9.832	***46.247***	7.189	46.725	7.582	**41.664**	7.769	283	51	253
Nighttime passenger cars before	43.169	9.708	45.407	7.433	46.148	6.538	46.774	5.869	59	27	31
Nighttime passenger cars 1st after	**38.763**	10.393	**53.684**	9.957	44.000	6.942	**41.029**	7.258	38	32	35
Nighttime passenger cars 2nd after	***41.874***	10.455	***47.310***	7.552	***49.063***	7.875	**42.333**	7.530	174	48	162
Opposed passenger cars before	41.056	7.565	42.944	10.067	43.400	4.881	46.750	4.070	18	10	12

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N1	N2	N3
Opposed passenger cars 1st after	40.273	10.612	**52.545**	6.743	43.545	6.362	**37.909**	7.368	11	11	11
Opposed passenger cars 2nd after	42.824	11.301	***47.235***	6.602	44.800	6.301	***43.605***	6.670	51	5	43
Unopposed passenger cars before	43.647	9.443	45.688	6.508	46.853	6.571	45.444	7.556	170	75	81
Unopposed passenger cars 1st after	**41.431**	8.961	**54.827**	8.855	**45.099**	6.666	**41.021**	7.129	202	172	187
Unopposed passenger cars 2nd after	42.264	9.917	***46.579***	7.431	***48.021***	7.840	**41.731**	7.767	406	94	372

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 110 shows a graphical representation of the speed profiles for Alabama Location #6. The general shape of the speed profiles for the three collection periods remained relatively similar.

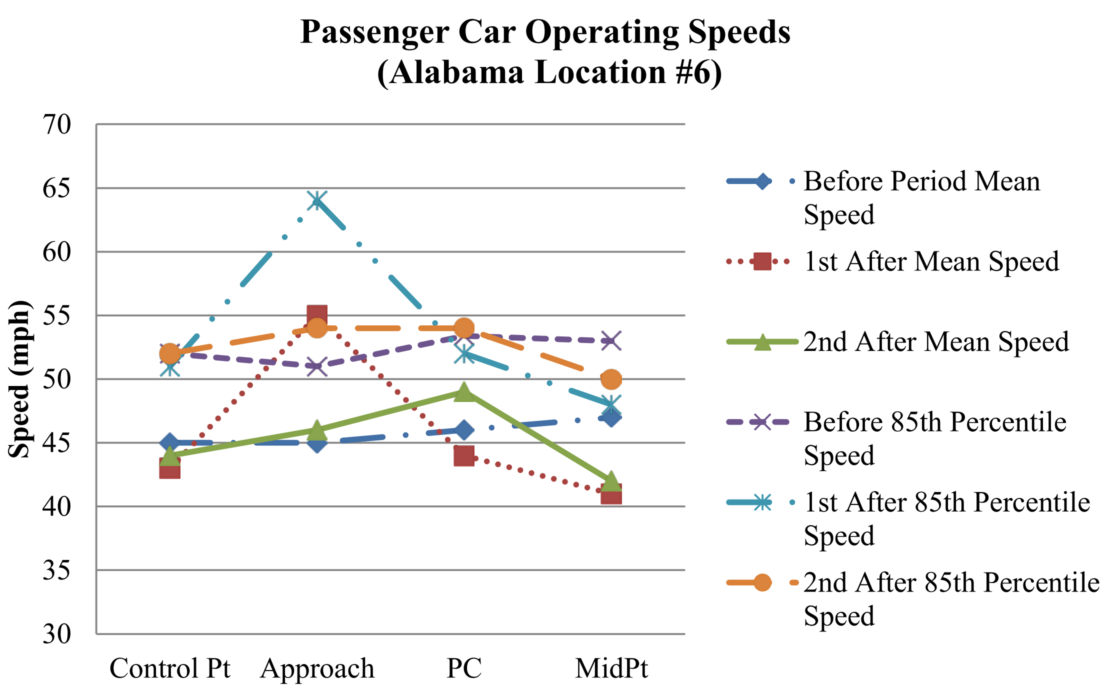


Figure 110. Graph. Operating speeds comparison at Alabama Location #6 (PSL = 40 mph).

Alabama Location #7

As table 70 shows, in the first after period at Alabama Location #7, operating speeds changed for many speed metrics. The only significant change at the control point was heavy truck speeds, which decreased by approximately 6.0 mph. Speeds at the approach decreased for all measured speed metrics. These speed decreases were between approximately 2.2 and 2.9 mph for passenger cars and 5.6 mph for heavy trucks. However, if the speed decrease at the control point was taken into account, this speed decrease was no longer significant. Speeds at the PC significantly decreased for the following speed metrics: all observations, passenger cars, daytime passenger cars, opposed passenger cars, and unopposed passenger cars. Speed decreases were practically insignificant, with speed decreases between approximately 0.7 and 1.3 mph. Speeds at the curve midpoint significantly decreased for all measured speed metrics. These speed decreases were between approximately 2.8 and 3.6 mph for passenger cars. Speeds for heavy trucks decreased by approximately 7.3 mph. However, if the speed decrease at the control point was taken into account, this speed decrease was no longer significant. After installing the OSBs, speeds initially decreased through the curve.

Many of the changes in the first after period were also present in the second after period. There were no significant speed changes at the control point. Speeds at the approach increased from the first after period, but speeds were still significantly lower than the before period for all speed metrics, except heavy trucks. Speed decreases were between approximately 1.4 and 1.7 mph. Speeds at the PC were similar to speeds in the first after period and were significantly lower for all passenger car speed metrics, except opposed passenger cars. These speed decreases were small, with decreases between approximately 0.8 and 1.2 mph. Passenger car speeds at the curve midpoint increased from the first after period, but they were all still lower than the before period, except opposed passenger car speeds were not significantly lower. These speed decreases were between approximately 1.2 and 1.9 mph.

Overall, speeds decreased at the approach, PC, and curve midpoint after the installation of the OSBs. However, these speed decreases were not practically significant.

Table 70. Before-after operating speeds at Alabama Location #7.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N1	N2	N3
All Observations before	53.159	9.499	43.648	5.400	36.950	4.293	38.819	5.257	429	422	419
All Observations 1st after	52.538	9.914	41.247	5.013	36.176	4.156	35.527	4.771	580	574	579
All Observations 2nd after	53.113	9.203	***42.260***	5.270	36.148	4.034	***37.234***	5.337	795	764	745
Passenger cars before	53.036	9.497	43.635	5.391	36.971	4.310	38.768	5.230	419	412	409
Passenger cars 1st after	52.546	9.918	41.341	4.992	36.249	4.150	35.594	4.777	560	554	559
Passenger cars 2nd after	53.127	9.141	***42.285***	5.234	36.164	4.048	***37.225***	5.259	780	751	730

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N1	N2	N3
Heavy trucks before	58.300	8.433	44.200	6.033	36.100	3.604	40.900	6.208	10	10	10
Heavy trucks 1st after	52.300	10.069	38.600	5.020	34.150	3.884	33.650	4.320	20	20	20
Heavy trucks 2nd after	52.400	12.362	41.000	7.000	35.231	3.059	***37.667***	8.566	15	13	15
Daytime passenger cars before	53.408	8.731	44.387	5.235	37.646	4.548	39.175	5.316	191	189	189
Daytime passenger cars 1st after	52.361	9.748	41.537	5.062	36.360	4.189	35.566	4.962	402	397	401
Daytime passenger cars 2nd after	53.165	9.232	***42.644***	5.289	36.448	4.154	***37.260***	5.336	514	500	481
Nighttime passenger cars before	52.724	10.104	43.004	5.450	36.399	4.020	38.418	5.141	228	223	220
Nighttime passenger cars 1st after	53.019	10.354	40.842	4.788	35.968	4.048	35.665	4.284	158	157	158
Nighttime passenger cars 2nd after	53.053	8.980	41.590	5.065	35.598	3.773	***37.157***	5.118	266	251	249
Opposed passenger cars before	52.148	8.668	43.241	5.701	36.657	4.310	38.355	4.989	108	105	107
Opposed passenger cars 1st after	51.773	10.502	41.000	4.558	35.625	3.671	35.042	4.632	97	96	96
Opposed passenger cars 2nd after	53.025	8.605	41.893	5.107	36.182	3.870	***37.514***	4.969	159	154	148
Unopposed passenger cars before	53.344	9.763	43.772	5.282	37.078	4.312	38.914	5.313	311	307	302
Unopposed passenger cars 1st after	52.708	9.795	41.413	5.079	36.380	4.235	35.708	4.803	463	458	463
Unopposed passenger cars 2nd after	53.153	9.280	***42.385***	5.266	36.159	4.096	***37.151***	5.332	621	597	582

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period

Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods

SD = Standard Deviation

PC = Point of Curvature

N_1 = Number of observations at the control point and approach

N_2 = Number of observations at the PC

N_3 = Number of observations at the midpoint

Figure 111 shows a graphical representation of the speed profiles for Alabama Location #7. The general shape of the speed profiles for the three collection periods remained relatively similar.

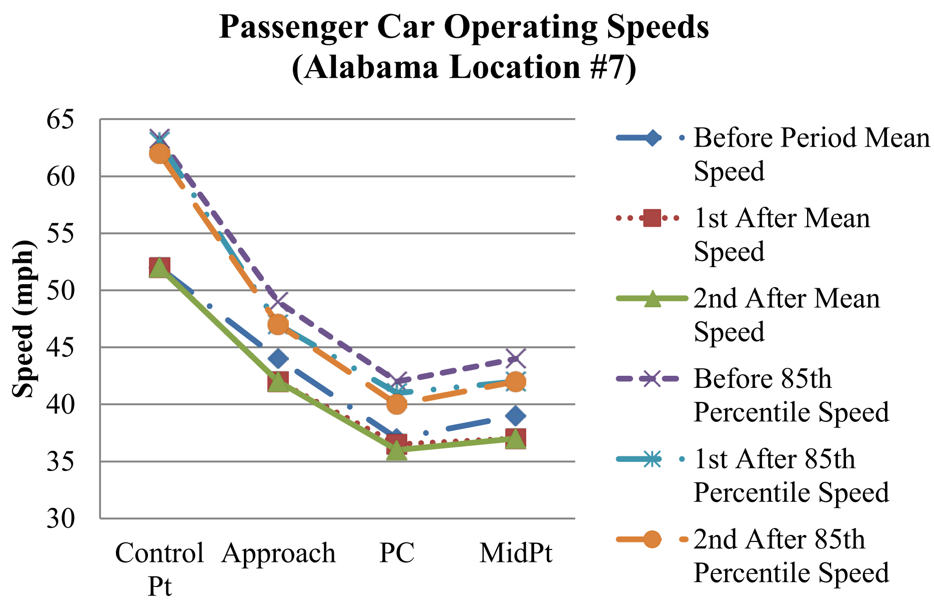


Figure 111. Graph. Operating speeds comparison at Alabama Location #7 (PSL = 35 mph).

Alabama Location #8

As table 71 shows, in the first after period at Alabama Location #8, operating speeds remained the same for most metrics. The only significant change at the control point was daytime passenger car speeds, which decreased by approximately 1.9 mph. There were no significant speed changes at the approach or PC for any speed metric. Speeds decreased at the curve midpoint for all speed metrics, except heavy trucks. The speed decreases were between approximately 2.7 and 3.5 mph. After the OSBs were installed, speeds significantly decreased at the curve midpoint during the first after period.

There were more speed changes in the second after period, but most these changes were from the first after period and not the before period. The only significant changes at the control point compared with the before period were speeds for all observations, passenger cars, and nighttime passenger cars increased, with increases of approximately 0.9, 0.9, and 1.2 mph, respectively. There were no significant speed changes at the approach when compared with the before period. Speeds did not significantly change at the PC, except heavy truck speeds increased by approximately 4.0 mph. Speeds at the curve midpoint decreased for the following speed metrics: all observations, passenger cars, daytime passenger cars, and unopposed passenger cars, with speed decreases of approximately 1.0, 1.0, 1.3, and 1.5 mph. Based on the second after period, speeds at the curve midpoint slightly decreased after applying the OSBs, but these speed reductions were not practical, with less than a 2.0-mph reduction.

Table 71. Before-after operating speeds at Alabama Location #8.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations before	41.540	6.029	44.644	8.437	40.657	5.331	42.239	6.340	376	341	310
All observations 1st after	41.095	6.122	43.958	6.022	40.503	4.697	**39.032**	5.118	379	370	375
All observations 2nd after	***42.467***	5.824	***45.129***	6.511	***41.091***	4.821	***41.275***	6.115	614	571	538
Passenger cars before	41.566	6.053	44.672	8.317	40.751	5.334	42.348	6.349	366	333	302
Passenger cars 1st after	41.108	6.173	43.919	5.967	40.560	4.701	**39.058**	5.138	369	361	365
Passenger cars 2nd after	***42.416***	5.734	***45.213***	6.434	***41.106***	4.798	***41.320***	6.144	587	548	515
Heavy trucks before	40.600	5.254	43.600	12.624	36.750	3.655	38.125	4.612	10	8	8
Heavy trucks 1st after	40.600	3.978	45.400	8.072	38.222	4.116	38.100	4.458	10	9	10
Heavy trucks 2nd after	43.593	7.582	43.296	7.927	**40.739**	5.429	40.261	5.446	27	23	23
Daytime passenger cars before	43.145	5.833	44.612	8.251	40.842	4.928	42.375	5.922	165	146	136
Daytime passenger cars 1st after	**41.245**	6.185	43.931	5.902	40.428	4.732	**38.951**	5.148	306	299	303
Daytime passenger cars 2nd after	***42.984***	5.802	***45.665***	6.153	40.865	4.563	***41.108***	6.125	367	347	334
Nighttime passenger cars before	40.269	5.934	44.721	8.391	40.679	5.642	42.325	6.696	201	187	166
Nighttime passenger cars 1st after	40.444	6.117	43.857	6.319	41.194	4.533	**39.581**	5.101	63	63	62
Nighttime passenger cars 2nd after	**41.468**	5.502	44.459	6.825	41.522	5.164	***41.713***	6.177	220	201	181
Opposed passenger cars before	41.750	6.708	42.841	7.546	40.823	6.354	42.000	7.255	88	79	71
Opposed passenger cars 1st after	40.991	6.438	42.616	5.091	39.773	4.692	**38.545**	4.781	112	110	110
Opposed passenger cars 2nd after	***42.582***	5.029	***45.277***	6.065	***40.983***	4.497	***42.173***	5.403	184	177	168

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
Unopposed passenger cars before	41.507	5.842	45.252	8.477	40.728	4.988	42.455	6.057	278	254	231
Unopposed passenger cars 1st after	41.160	6.066	44.486	6.235	40.904	4.673	**39.278**	5.279	257	251	255
Unopposed passenger cars 2nd after	***42.340***	6.032	45.184	6.603	41.164	4.940	***40.908***	6.439	403	371	347

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
SD = Standard Deviation
PC = Point of Curvature
N1 = Number of observations at the control point and approach
N2 = Number of observations at the PC
N3 = Number of observations at the midpoint

Figure 112 shows a graphical representation of the speed profiles for Alabama Location #8. The general shape of the speed profiles for the three collection periods remained relatively similar.

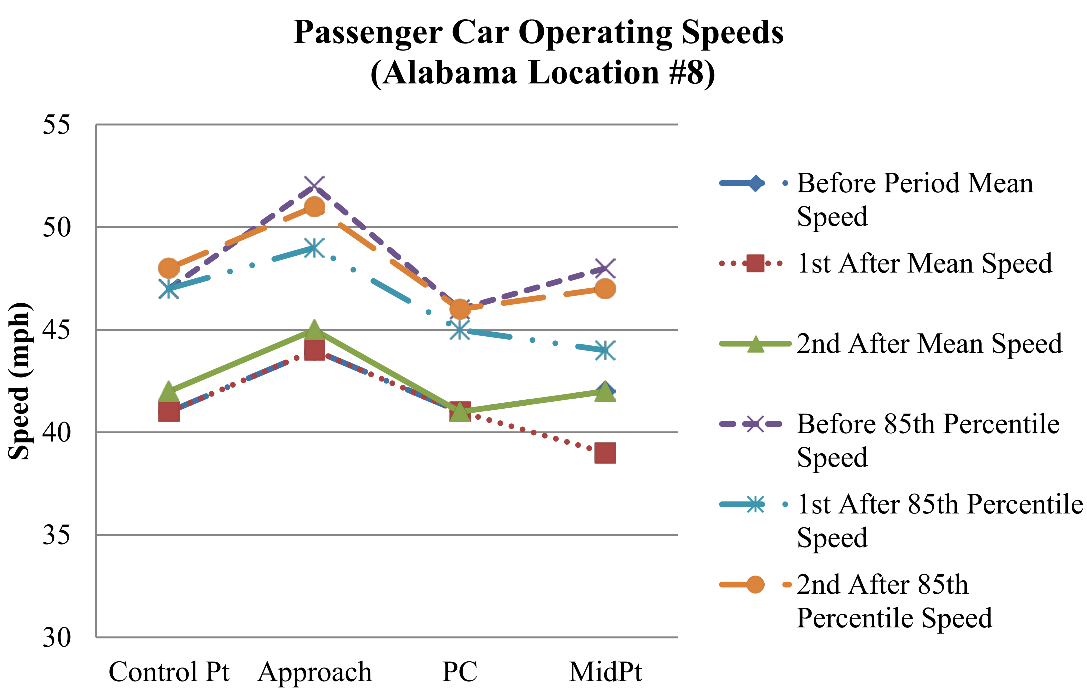


Figure 112. Graph. Operating speeds comparison at Alabama Location #8 (PSL = 35 mph).

Alabama Summary of Results

Overall, the optical speed bars had no effect on operating speeds at the Alabama sites. There were no changes in speeds that were consistent across all sites.

Massachusetts

This section of the report presents the before-after comparisons for the seven Massachusetts OSB sites.

Southbound Tucker Road, MA

As table 72 shows, operating speeds changed significantly at the southbound Tucker Road site in the after period. Oddly, speeds at the control point decreased between approximately 5.6 and 7.3 mph for all passenger car speed metrics, and approximately 9.4 mph for heavy trucks. This difference in operating speeds at the control point cannot be explained, except it may be a result of a measurement error caused by the on-pavement sensor. Speeds in the after period remained relatively constant from the control point to the approach. Speeds increased slightly at the approach for the following speed metrics: all observations, passenger cars, daytime passenger cars, and unopposed passenger cars, with speed increases of approximately 0.8, 0.9, 1.2, and 1.1 mph, respectively. These speed increases were minimal and were not practically significant. Speeds also increased at the PC for all measured speed metrics, with speeds increasing between approximately 3.0 and 3.7 mph for all passenger car speed metrics, and approximately 4.3 mph for heavy trucks. Speeds in the first after period remained relatively constant from the control point to the approach. There was a sensor malfunction at the curve midpoint, and no speeds were collected and no *t*-test was performed. Speeds at the PC increased between 3.0 and 3.7 mph for passenger cars after the OSBs were installed at this site.

Table 72. Before-after operating speeds at southbound Tucker Road.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N1	N2	N3
All observations before	44.458	7.485	36.492	5.228	35.223	4.467	35.329	5.014	577	539	514
All observations after	**37.651**	5.164	**37.337**	5.164	**38.547**	6.650	0.000	0.000	341	139	0
Passenger cars before	44.466	7.510	36.506	5.236	35.241	4.475	35.334	5.022	573	535	512
Passenger cars after	**37.753**	5.147	**37.428**	5.169	**38.569**	6.692	0.000	0.000	332	137	0
Heavy trucks before	43.250	0.957	34.500	3.873	32.750	2.500	34.000	1.414	4	4	2
Heavy trucks after	**33.889**	4.567	34.000	3.873	**37.000**	2.828	0.000	0.000	9	2	0
Daytime passenger cars before	45.136	6.983	36.782	5.064	35.488	4.127	35.420	4.809	316	293	286
Daytime passenger cars after	**38.616**	5.033	**37.966**	5.145	**38.493**	7.690	0.000	0.000	203	71	0
Nighttime passenger cars before	43.642	8.048	36.167	5.431	34.942	4.855	35.226	5.289	257	242	226
Nighttime passenger cars after	**36.395**	5.049	36.581	5.114	**38.652**	5.476	0.000	0.000	129	66	0
Opposed passenger cars before	44.306	7.210	36.972	5.543	35.272	3.792	35.036	5.203	252	235	224
Opposed passenger cars after	**38.686**	5.503	37.822	5.583	**38.390**	8.812	0.000	0.000	118	41	0
Unopposed passenger cars before	44.592	7.745	36.140	4.960	35.217	4.951	35.566	4.874	321	300	288
Unopposed passenger cars after	**37.238**	4.876	**37.210**	4.926	**38.646**	5.603	0.000	0.000	214	96	0

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 113 shows a graphical representation of the speed profiles for southbound Tucker Road. In the after period, mean and 85th percentile speeds decreased significantly at the control point while they increased at the PC. Midpoint speeds were not available because of a sensor malfunction.

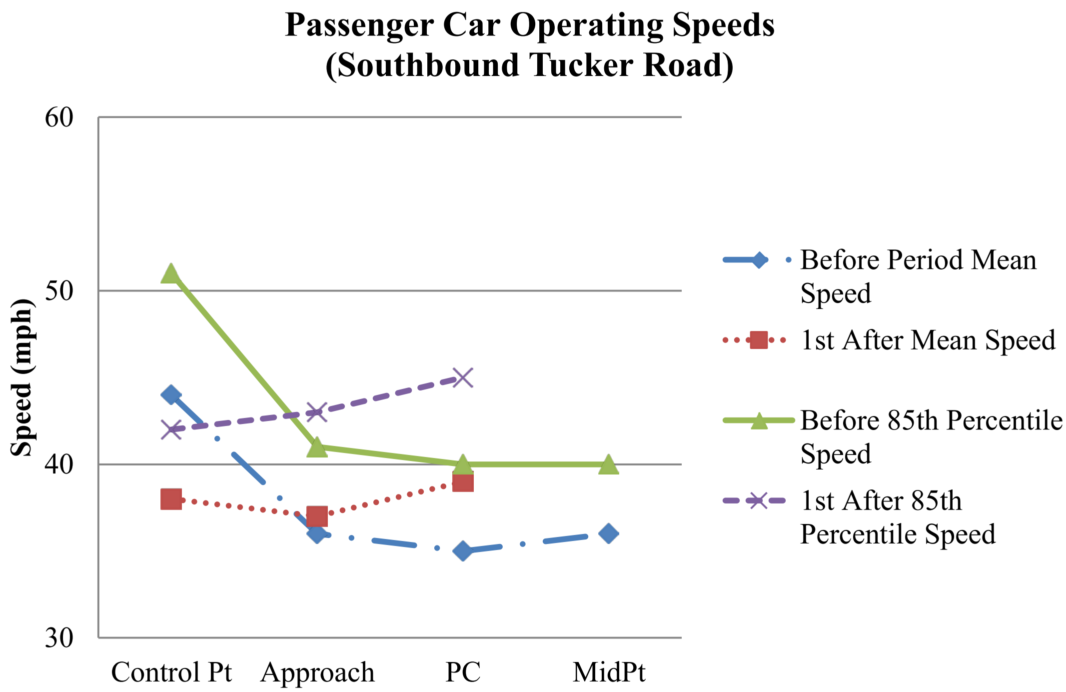


Figure 113. Graph. Operating speeds comparison at southbound Tucker Road (PSL = 30 mph).

Northbound Tucker Road, MA

As table 73 shows, operating speeds significantly changed at northbound Tucker Road, but the changes did not follow a clear trend. Speeds at the control point did not change in the first after period. Speeds decreased slightly at the approach for the following speed metrics: all observations, heavy trucks, daytime passenger cars, and opposed passenger cars, with speed decreases of approximately 0.8, 0.7, 0.7, and 0.9 mph, respectively. These changes in speeds were minimal and not practically significant. Speeds at the PC increased between approximately 1.8 and 3.1 mph for all speed metrics. However, speeds at the curve then decreased for all speed metrics, except heavy trucks. Speeds at the curve midpoint decreased between approximately 1.1 and 1.7 mph. There was no strong trend in the effects of the OSBs at this site, with speeds significantly increasing at the PC but then decreasing at the curve midpoint.

Table 73. Before-after operating speeds at northbound Tucker Road.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N1	N2	N3
All observations before	33.895	4.361	37.609	4.247	33.381	3.820	35.507	4.852	573	565	458
All observations after	33.761	4.403	**36.850**	5.004	**35.768**	4.249	**33.904**	5.674	394	367	355
Passenger cars before	33.919	4.361	37.633	4.251	33.403	3.820	35.525	4.858	569	561	455
Passenger cars after	33.903	4.347	**36.911**	5.037	**35.821**	4.230	**33.980**	5.645	383	358	348
Heavy trucks before	30.500	3.000	34.250	1.500	30.250	2.500	32.667	3.055	4	4	3
Heavy trucks after	28.818	3.516	34.727	3.133	**33.667**	4.743	30.143	6.283	11	9	7
Daytime passenger cars before	34.135	4.263	37.768	4.219	33.683	3.551	36.145	4.549	341	338	276
Daytime passenger cars after	34.416	3.787	**37.024**	4.764	**36.771**	3.545	**34.455**	5.391	209	192	187
Nighttime passenger cars before	33.596	4.495	37.430	4.300	32.978	4.167	34.570	5.168	228	223	179
Nighttime passenger cars after	33.287	4.877	36.776	5.358	**34.723**	4.680	**33.429**	5.896	174	166	161
Opposed passenger cars before	33.681	4.172	37.453	4.207	33.369	3.635	35.839	4.758	351	347	280
Opposed passenger cars after	34.126	4.032	**36.581**	4.621	**36.017**	3.937	**34.320**	4.864	191	179	172
Unopposed passenger cars before	34.303	4.634	37.922	4.315	33.458	4.111	35.023	4.986	218	214	175
Unopposed passenger cars after	33.682	4.640	37.240	5.412	**35.626**	4.506	**33.648**	6.313	192	179	176

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 114 shows a graphical representation of the speed profiles for northbound Tucker Road. In the after period, the mean and 85th percentile speeds increased slightly at the PC and midpoints. The control point and approach speeds remained relatively similar during both the before and after data-collection periods.

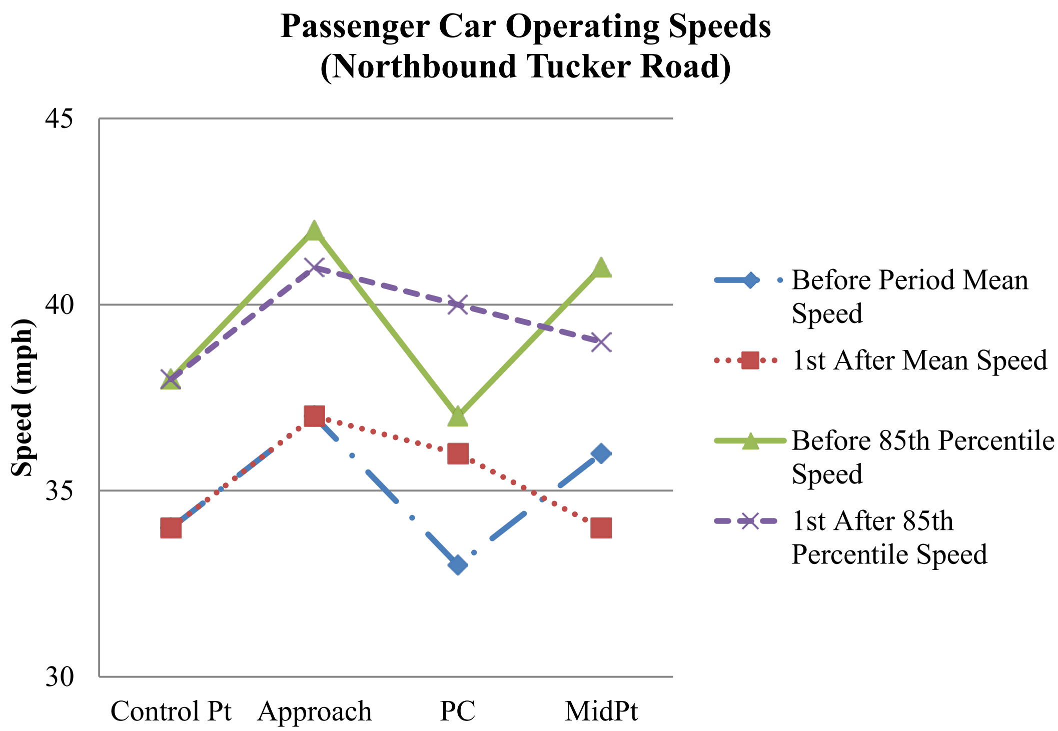


Figure 114. Graph. Operating speeds comparison at northbound Tucker Road (PSL = 35 mph).

Southbound Reed Road, Massachusetts

As table 74 shows, operating speeds changed significantly at the southbound Reed Road site in the first after period. Operating speeds decreased slightly at the control point for the following speed metrics: all observations, passenger cars, nighttime passenger cars, opposed passenger cars, and unopposed passenger cars, with speed decreases of approximately 1.2, 1.2, 2.7, 1.2, and 1.1 mph, respectively. These speed decreases were not practically significant, except for the nighttime speed decrease of 2.7 mph. There was a sensor malfunction at the approach, so no speeds were collected, but the research team performed a *t*-test. Speeds increased significantly for all passenger car speed metrics at the PC, with speed increases between approximately 1.4 and 3.8 mph. These speed increases become more significant and larger in magnitude if the speed decrease at the control point was taken into account. However, speeds were then significantly lower at the curve midpoint for all passenger car speed metrics. These speed decreases were approximately 2.7 to 4 mph. There was no clear trend in operating speeds at this site after the OSBs were installed. Operating speeds increased at the PC in the after period, but then operating speeds decreased at the curve midpoint.

Table 74. Before-after operating speeds at southbound Reed Road.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations before	39.966	5.426	39.382	6.015	38.826	4.288	39.363	4.990	775	482	590
All observations after	**38.723**	4.995	0.000	0.000	**41.525**	6.728	**36.190**	5.133	328	324	327
Passenger cars before	40.036	5.395	39.450	5.985	38.811	4.227	39.416	4.945	756	471	575
Passenger cars after	**38.831**	4.957	0.000	0.000	**41.541**	6.732	**36.232**	5.111	320	316	319
Heavy trucks before	37.211	6.070	36.684	6.758	39.455	6.654	37.333	6.355	19	11	15
Heavy trucks after	34.375	4.868	0.000	0.000	40.875	6.978	34.500	6.094	8	8	8
Daytime passenger cars before	39.754	5.282	39.249	6.262	38.558	3.773	39.692	4.767	337	224	247
Daytime passenger cars after	39.691	4.417	0.000	0.000	**42.326**	7.233	**36.963**	4.597	188	184	187
Nighttime passenger cars before	40.263	5.481	39.611	5.754	39.040	4.596	39.207	5.072	419	247	328
Nighttime passenger cars after	**37.606**	5.422	0.000	0.000	**40.447**	5.816	**35.197**	5.618	132	132	132
Opposed passenger cars before	39.971	5.365	39.753	6.653	38.669	3.911	39.469	4.916	377	242	275
Opposed passenger cars after	**38.781**	4.926	0.000	0.000	**41.372**	6.445	**36.445**	4.910	228	226	227
Unopposed passenger cars before	40.100	5.432	39.148	5.227	38.961	4.542	39.367	4.979	379	229	300
Unopposed passenger cars after	**38.957**	5.056	0.000	0.000	**41.967**	7.427	**35.707**	5.570	92	90	92

Bold indicates significance at 95 percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 115 shows a graphical representation of the speed profiles for southbound Reed Road. In the after period, mean speeds increased at the midpoint while 85th percentile speeds increased at the PC point. Approach speeds were not available because of a malfunctioning sensor.

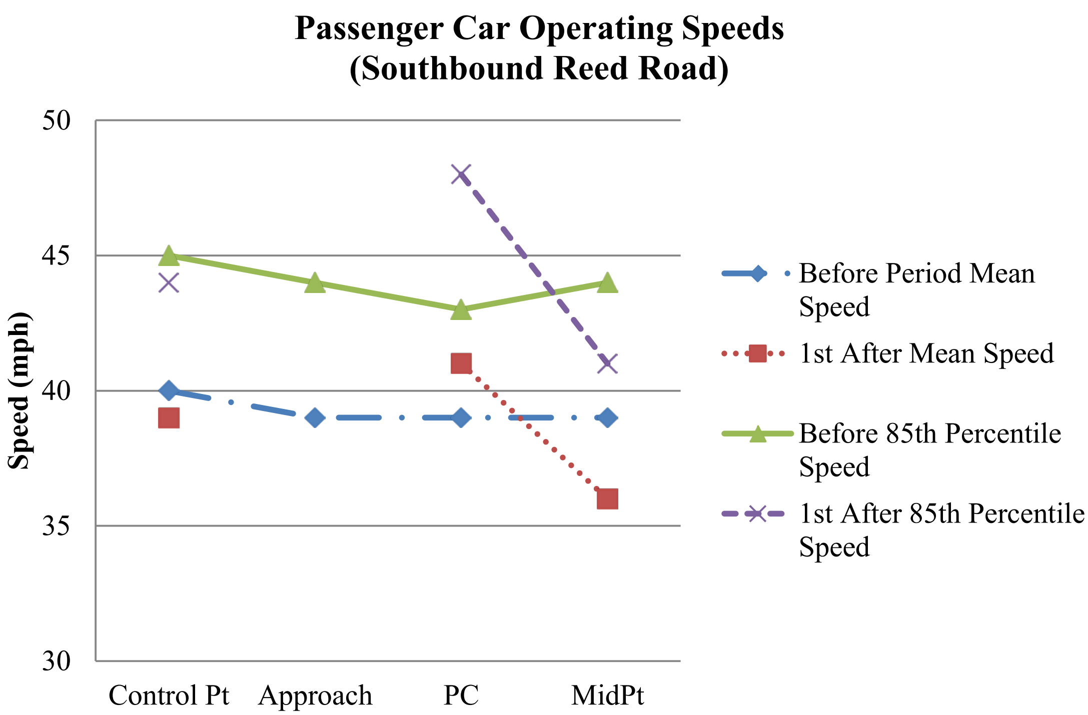


Figure 115. Graph. Operating speeds comparison at southbound Reed Road (PSL = 25 mph).

Southbound New Boston Road, MA

Overall, operating speeds were higher in the after period at southbound New Boston Road. As table 75 shows, operating speeds increased for all passenger car speed metrics, with speed increases between approximately 1.3 to 4.2 mph. Speeds increased at the approach for the following speed metrics: all observations, passenger cars, heavy trucks, and opposed passenger cars, with speed increases of approximately 1.0, 0.9, 6.1, and 3.2 mph, respectively. However, if the speed increase at the control point was taken into account, speeds at the approach for all observations and passenger cars actually decreased slightly. Speeds increased at the PC for all passenger car speed metrics, with speed increases between approximately 1.6 and 2.7 mph. However, if the speed increase at the control point was taken into accounted, the speeds at the PC did not change from the before period for any passenger car speed metric. Speeds at the curve midpoint increased for all speed metrics, with speed increases between approximately 0.9 and 2.6 mph for passenger cars and 5.6 mph for heavy trucks. Once again, none of these speed increases for passenger cars were significant if the speed increase at the control point was taken into accounted. Operating speeds were higher at southbound New Boston Road in the after period, but the OSBs had no effect on speeds after accounting for higher speeds at the control point.

Table 75. Before-after operating speeds at southbound New Boston Road.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations before	37.004	6.532	37.704	5.967	37.489	5.607	36.624	6.206	257	229	229
All observations after	**39.003**	6.359	**38.675**	5.154	**39.345**	5.446	**38.025**	6.192	381	293	325
Passenger cars before	36.968	6.565	37.766	5.980	37.465	5.636	36.656	6.224	252	226	227
Passenger cars after	**38.962**	6.366	**38.630**	5.160	**39.323**	5.470	**38.013**	6.211	373	288	318
Heavy trucks before	38.800	4.712	34.600	4.722	39.333	2.082	33.000	0.000	5	3	2
Heavy trucks after	40.875	6.081	**40.750**	4.713	40.600	4.037	**38.571**	5.653	8	5	7
Daytime passenger cars before	36.690	5.930	37.655	5.583	37.576	4.603	36.760	5.551	113	99	100
Daytime passenger cars after	**38.567**	6.228	38.424	5.389	**39.403**	5.390	**38.073**	6.152	210	154	178
Nighttime passenger cars before	37.194	7.052	37.856	6.302	37.378	6.343	36.575	6.728	139	127	127
Nighttime passenger cars after	**39.472**	6.525	38.896	4.852	**39.231**	5.579	**37.936**	6.307	163	134	140
Opposed passenger cars before	35.638	5.307	36.259	5.872	36.788	5.410	35.962	6.145	58	52	52
Opposed passenger cars after	**39.796**	6.541	**39.500**	5.061	**39.500**	5.780	**38.525**	6.375	108	86	99
Unopposed passenger cars before	37.366	6.859	38.216	5.953	37.667	5.702	36.863	6.250	194	174	175
Unopposed passenger cars after	**38.623**	6.275	38.275	5.167	**39.248**	5.346	37.781	6.136	265	202	219

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 116 shows a graphical representation of the speed profiles for southbound New Boston Road. In the after period, 85th percentile speeds increased at the PC and midpoint. All other speed profiles remained relatively similar in the before and after data-collection periods.

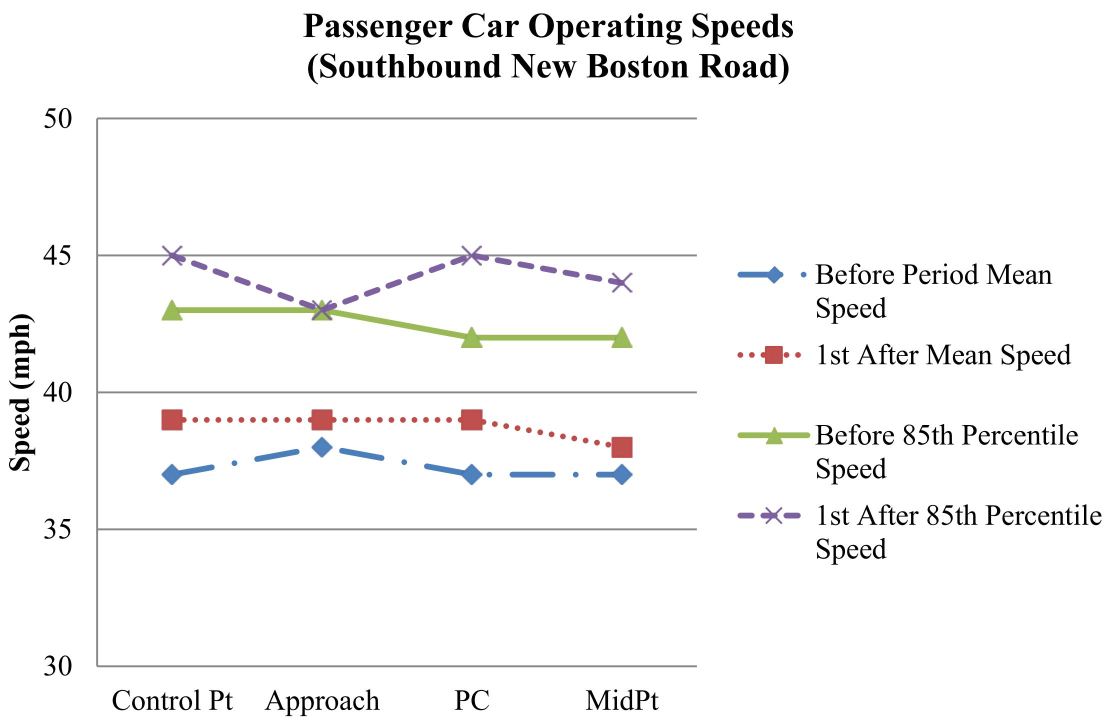


Figure 116. Graph. Operating speeds comparison on southbound New Boston Road (PSL = 35 mph).

Northbound New Boston Road, MA

As table 76 shows, operating speeds changed significantly at the northbound New Boston Road site in the after period. All observations and passenger car speeds increased at the control point by approximately 1.0 and 0.9 mph, respectively. These slight speed increases do not have any practical significance. Approach speeds increased for the following speed metrics: all observations, passenger cars, heavy trucks, nighttime passenger cars, and unopposed passenger cars, with speed increases of approximately 1.5, 1.4, 6.8, 2.3, and 1.5 mph, respectively. Speeds at the PC did not change in the after period. Speeds at the curve midpoint increased between approximately 2.8 and 4.5 mph for all speed metrics, except heavy trucks. Speeds significantly increased at the curve midpoint after the application of the OSBs at this site. Overall, the installation of the OSBs did not result in reduced operating speed at this site.

Table 76. Before-after operating speeds at northbound New Boston Road.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations before	39.796	5.704	36.876	5.691	39.500	4.955	33.742	6.553	225	114	209
All observations after	**40.769**	7.248	**38.356**	6.573	39.559	6.720	**37.568**	7.593	320	118	229
Passenger cars before	39.857	5.680	36.928	5.686	39.558	4.939	33.720	6.563	223	113	207
Passenger cars after	**40.780**	7.267	**38.366**	6.555	39.535	6.781	**37.546**	7.608	314	114	227
Heavy trucks before	33.000	5.657	31.000	2.828	33.000	0.000	36.000	7.071	2	1	2
Heavy trucks after	40.167	6.706	**37.833**	8.159	40.250	5.315	40.000	7.071	6	4	2
Daytime passenger cars before	40.074	5.352	37.475	5.801	40.066	4.694	34.627	6.531	122	61	110
Daytime passenger cars after	40.719	7.186	38.270	6.268	39.134	6.080	**37.443**	7.915	196	67	131
Nighttime passenger cars before	39.594	6.070	36.267	5.499	38.962	5.194	32.691	6.478	101	52	97
Nighttime passenger cars after	40.881	7.430	**38.525**	7.031	40.106	7.704	**37.688**	7.207	118	47	96
Opposed passenger cars before	40.119	4.835	36.810	4.830	39.714	3.989	32.789	4.911	42	21	38
Opposed passenger cars after	40.662	8.230	37.954	5.792	38.048	6.561	**37.289**	7.884	65	21	38
Unopposed passenger cars before	39.796	5.870	36.956	5.878	39.522	5.149	33.929	6.874	181	92	169
Unopposed passenger cars after	40.811	7.012	**38.474**	6.747	39.871	6.819	**37.598**	7.572	249	93	189

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 117 shows a graphical representation of the speed profiles for northbound New Boston Road. Mean and 85th percentile speeds increased at the midpoint in the after period. The general shape of the speed profiles at all other locations remained relatively similar.

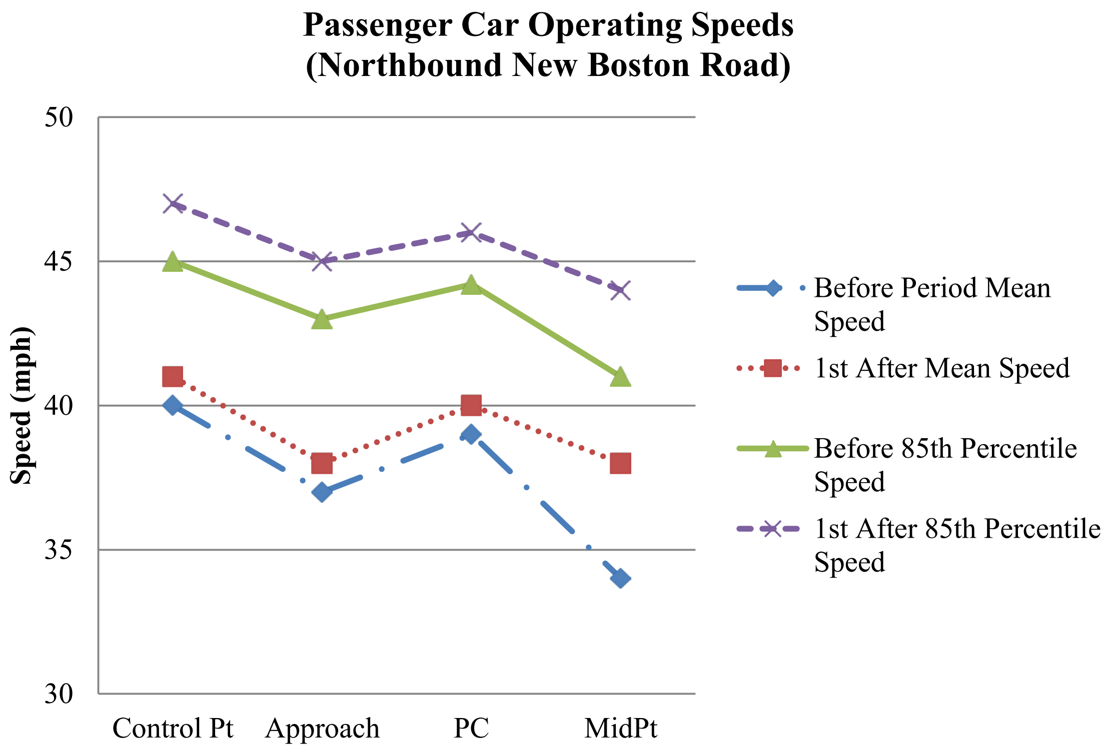


Figure 117. Graph. Operating speeds comparison on northbound New Boston Road (PSL = 35 mph).

Southbound Braley Hill Road, MA

As table 77 shows, operating speeds changed significantly at the southbound Braley Hill Road site in the after period. At the control point, daytime passenger car speeds increased by approximately 1.2 mph, and nighttime passenger car speeds decreased by approximately 1.7 mph. Speeds at the approach were significantly higher for all speed metrics, with increases between 5.6 and 7.3 mph for the different passenger car speed metrics and 11.6 mph for heavy trucks. The speeds at the approach were approximately 5 to 6 mph higher than speeds at the control point in the after period. This does not reflect how drivers typically drive, so this significant increase at the approach may have resulted from a measurement error from the on-pavement sensor. Speeds then decreased at the PC for the following speed metrics: all observations, passenger cards, nighttime passenger cars, opposed passenger cars, and unopposed passenger cars, with speed decreases of approximately 1.2, 1.2, 1.6, 1.9, and 1.0 mph, respectively. The speed decreases at the PC were small in magnitude and not practically significant. Speeds at the curve midpoint also decreased in the after period for the following speed metrics: all observations, passenger cars, daytime passenger cars, nighttime passenger cars, and unopposed passenger cars, with speed decreases of approximately 2.2, 2.3. 1.7, 2.8, and 2.5 mph, respectively. Speeds throughout the curve decreased between 1 and 3 mph at this site after the application of the OSBs; this did not consider the significant increase at the approach speed because that appears to be the result of a measurement error.

Table 77. Before-after operating speeds at southbound Braley Hill Road.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N1	N2	N3
All observations before	39.360	5.097	38.508	5.835	34.585	4.791	36.365	6.947	317	294	252
All observations after	39.040	5.667	**45.191**	7.031	**33.415**	4.160	**34.176**	7.407	277	260	256
Passenger cars before	39.472	5.136	38.600	5.872	34.665	4.693	36.502	6.905	305	284	243
Passenger cars after	39.208	5.645	**45.075**	6.760	**33.520**	4.174	**34.244**	7.497	265	250	246
Heavy trucks before	36.500	2.844	36.167	4.345	32.300	6.977	32.667	7.483	12	10	9
Heavy trucks after	35.333	5.033	**47.750**	11.671	30.800	2.821	32.500	4.577	12	10	10
Daytime passenger cars before	38.640	4.423	37.953	4.997	34.433	4.591	35.697	7.398	150	141	122
Daytime passenger cars after	**39.801**	4.950	**45.298**	6.401	33.701	4.205	**34.038**	7.345	141	134	132
Nighttime passenger cars before	40.277	5.640	39.226	6.566	34.895	4.797	37.314	6.297	155	143	121
Nighttime passenger cars after	**38.532**	6.297	**44.823**	7.164	**33.310**	4.146	**34.482**	7.696	124	116	114
Opposed passenger cars before	39.211	4.485	37.958	5.247	34.758	4.736	35.185	6.355	71	66	54
Opposed passenger cars after	38.981	5.634	**44.096**	7.604	**32.837**	4.520	33.820	7.176	52	49	50
Unopposed passenger cars before	39.551	5.324	38.795	6.046	34.638	4.691	36.878	7.025	234	218	189
Unopposed passenger cars after	39.263	5.660	**45.315**	6.534	**33.687**	4.080	**34.352**	7.591	213	201	196

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 118 shows a graphical representation of the speed profiles for southbound Braley Hill Road. In the after period, the mean and 85th percentile speeds increased significantly at the PC point. Speed profiles at all other points remained relatively similar during both the before and after data-collection periods.

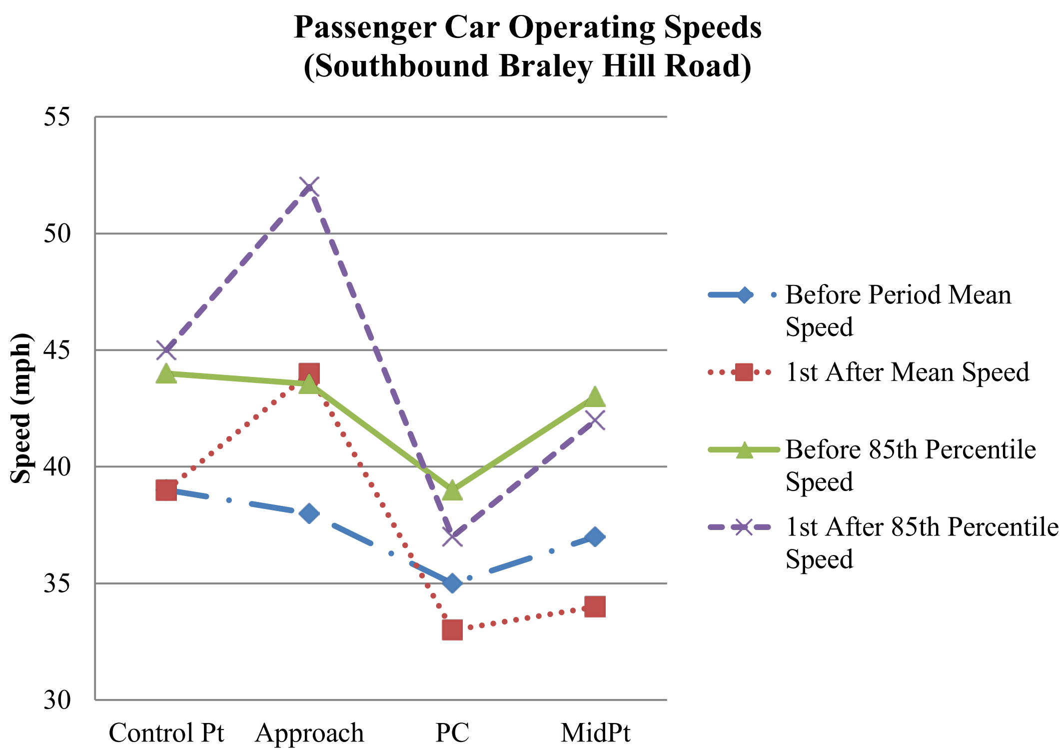


Figure 118. Graph. Operating speeds comparison at southbound Braley Hill Road (PSL = 30 mph).

Northbound Braley Hill Road, MA

As table 78 shows, operating speeds changed significantly at the northbound Braley Hill Road site in the after period. Oddly, speeds at the control point decreased between approximately 6.1 and 7.8 mph for all speed metrics, except heavy trucks. Heavy truck speeds decreased by approximately 9.2 mph, but this was not significant because the SD was so high and there were few observations in both periods. This difference in operating speeds at the control point cannot be explained, except that it may be a result of a measurement error caused by the on-pavement sensor. Speeds at the approach did not change for any speed metric if the significant reductions in speeds at the control point were not taken into account. Speeds at the PC increased between approximately 2.8 and 4 mph for all speed metrics. These speed increases at the PC increase in magnitude and become more significant if the speed decreases at the control were taken into accounted. Speeds at the curve midpoint also increased between approximately 1.0 and 1.4 mph for all speed metrics, except heavy trucks. Heavy truck speeds decreased by approximately 2.7 mph, but this was not significant because there were few observations in both periods. These speed increases at the curve midpoint increased in magnitude and become more significant if the speed decreases at the control were taken into account. Based on the first after period, speeds at the PC and curve midpoint significantly increased after the application of the OSBs. The speed increase at the PC was higher than at the curve midpoint, with the speed increase at the curve midpoint not being practically significant at a maximum speed increase of 1.4 mph for all speed metrics.

Table 78. Before-after operating speeds at northbound Braley Hill Road.

Speed Metric	Control Point Speed	Control Point SD	Approach Speed	Approach SD	PC Speed	PC SD	Mid Speed	Mid SD	N_1	N_2	N_3
All observations before	44.188	7.842	38.650	4.290	37.565	3.981	38.530	3.990	277	271	268
All observations after	**37.166**	6.209	38.456	4.774	**40.500**	4.758	**39.719**	4.198	331	224	320
Passenger cars before	44.169	7.738	38.692	4.294	37.603	3.993	38.572	3.992	273	267	264
Passenger cars after	**37.185**	6.201	38.469	4.759	**40.541**	4.795	**39.748**	4.168	324	218	313
Heavy trucks before	45.500	15.022	35.750	3.096	35.000	1.826	35.750	3.096	4	4	4
Heavy trucks after	36.286	7.017	37.857	5.815	**39.000**	2.966	38.429	5.623	7	6	7
Daytime passenger cars before	44.238	6.538	38.524	4.084	37.431	3.703	38.565	3.823	147	144	147
Daytime passenger cars after	**37.994**	5.366	38.893	4.567	**40.518**	4.922	**39.921**	4.115	169	137	164
Nighttime passenger cars before	44.087	8.965	38.889	4.536	37.805	4.315	38.581	4.210	126	123	117
Nighttime passenger cars after	**36.303**	6.910	38.006	4.934	**40.580**	4.604	**39.557**	4.230	155	81	149
Opposed passenger cars before	43.794	7.456	38.730	4.178	36.983	4.312	38.381	4.046	63	60	63
Opposed passenger cars after	**37.703**	5.891	38.099	3.981	**40.457**	5.413	**39.644**	4.193	91	70	90
Unopposed passenger cars before	44.281	7.835	38.681	4.338	37.783	3.889	38.632	3.983	210	207	201
Unopposed passenger cars after	**36.983**	6.319	38.614	5.031	**40.581**	4.493	**39.789**	4.166	233	148	223

Bold indicates significance at the 95-percent confidence level (α=0.05)
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Figure 119 shows a graphical representation of the speed profiles for northbound Braley Hill Road. The mean and 85th percentile control point speeds decreased in the after period while speed profiles at all points remained relatively similar in both the before and after period.

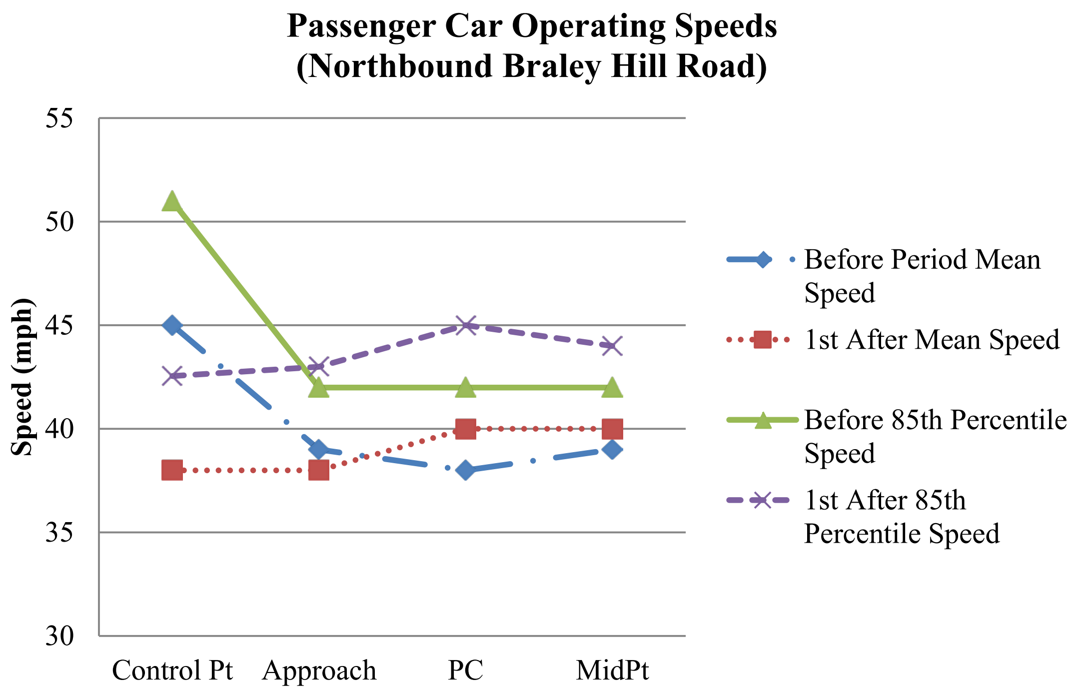


Figure 119. Graph. Operating speeds comparison at northbound Braley Hill Road (PSL = 40 mph).

Massachusetts Results Summary

Overall, the OSBs did not have an effect on operating speeds at the Massachusetts sites. There were no changes in speeds that were consistent across all of the sites.

Speed Difference Analysis

The next measures computed using the speed data were the change in speed from the approach to PC and the change in speed from the control point to the midpoint of the horizontal curve. These measures were taken in the before and both after periods at each of the treatment sites. As discussed previously, the research team used a *t*-test for independent samples to compare the two after periods with the before period and to each other. A positive value of delta speed indicates that a mean decrease in speed occurred from the approach to PC or from the control point to the curve midpoint. A negative value represents a mean increase in the speed difference between the two locations.

Arizona

Table 79 shows the change in speed (delta) from the approach to the PC and from the control point to the curve midpoint.

Table 79. Change in speeds for Arizona sites.

Speed Metric	Delta Speed (Approach to PC)	Standard Deviation	Delta Speed (Control Point to Midpoint)	Standard Deviation	N_1	N_2
NB Pierce Ferry Road before	0.267	4.527	10.514	7.776	90	107
NB Pierce Ferry Road 1st after	1.059	5.513	**12.806**	8.689	102	180
NB Pierce Ferry Road 2nd after	***5.733***	4.654	***5.392***	7.379	161	148
SB County Route 1 before	4.142	3.903	8.907	9.947	141	129
SB County Route 1 1st after	4.243	4.167	**12.123**	10.490	169	154
SB County Route 1 2nd after	***2.216***	3.438	***5.183***	10.640	97	60
SB Diamond Bar Road before	-1.535	4.277	3.152	7.979	99	99
SB Diamond Bar Road 1st after	-1.676	5.662	2.031	9.751	37	64
SB Diamond Bar Road 2nd after	***1.547***	3.856	***11.600***	8.375	53	105
SB Shinarump Road before	4.656	6.367	-3.973	7.219	90	73
SB Shinarump Road 1st after	4.043	4.798	-3.056	6.111	93	72
SB Shinarump Road 2nd after	3.149	6.500	***3.566***	9.846	101	136

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
NB = Northbound
SB = Southbound
PC = Point of Curvature
N_1 = Number of observations approach to PC
N_2 = Number of observations control point to midpoint

As table 79 shows, there was no difference in delta speed from the approach to the PC for any sites in the first after period. There was a significant increase in delta speed (control point to midpoint) of approximately 2.3 mph on northbound Pierce Ferry Road in the first after period. This was a result of vehicles having a slower speed at the curve midpoint in the first after period. There was also a significant increase in delta speed (control point to midpoint) of approximately 3.2 mph on southbound County Route 1. This was a result of vehicles travelling faster at the control point in the first after period.

There were more significant changes in delta speed in the second after period than there were in the first after period, with delta speed changing at every site. Delta speed (approach to PC) from the approach to the PC increased by approximately 5.5 mph on northbound Pierce Ferry Road, which was a result of higher speeds at the PC in the second after period. Conversely, delta speed (control point to midpoint) decreased by approximately 5.1 mph on northbound Pierce Ferry Road, which was a result of higher speeds at the curve midpoint in the second after period. Speeds significantly decreased at this site at the PC on northbound Pierce Ferry Road, but then they significantly increased at the curve midpoint in the second after period. Delta speed (approach to PC) decreased by approximately 1.9 mph on southbound County Route 1, which was a result of higher speeds at the curve approach, but even higher speeds at the PC. Delta speed (control point to midpoint) also decreased by approximately 3.7 mph on southbound County Route 1, which resulted from vehicles accelerating out of the curve. Delta speed (approach to PC) was actually negative in the before period on southbound Diamond Bar Road, representing speeds increased from the approach to PC. However, delta speed (approach to PC) was positive in the second after period at this site. There was a significant increase of approximately 3.1 mph in delta speed, which was a result of vehicles accelerating slightly from the control point to the approach in the second after period. Delta speed (control point to midpoint) changed drastically in the second after period on southbound Diamond Bar Road. Delta speed increased by approximately 8.4 mph, which was a result of vehicles decelerating at a much greater rate from the PC to the curve midpoint in the second after period. Finally, delta speed (control point to midpoint) was actually negative in the before period, but delta speed was positive in the second after period. Delta speed (control point to midpoint) increased by approximately 7.5 mph on southbound Shinarump Road, which resulted from vehicles decelerating from the PC to the curve midpoint in the second after period, where vehicles accelerated from the PC to the curve midpoint in the before period.

Overall, delta speed did not change in the first after period after the OSBs were installed at the Arizona sites, but delta speeds were significantly different at all sites in the second after period. However, there was no clear trend in the change in delta speed as a result of installing the OSBs. Delta speed from the PC to the approach increased by approximately 5.5 and 3.1 mph at two sites, while delta speed decreased by approximately 1.9 mph at another site. Delta speed from the control point to the curve midpoint decreased by approximately 5.1 and 3.7 mph at two sites, while delta speed increased by approximately 8.4 and 7.5 mph at two other sites. The change in delta speed varied both in direction and magnitude across the different Arizona sites, with no strong trend present. Overall, the OSBs did not have an effect on operating speeds at the Arizona sites. There were no changes in speeds that were consistent across all of the sites.

Alabama

Table 80 shows the change in speed (delta) from the approach to the PC and from the control point to the curve midpoint.

Table 80. Change in speeds for Alabama sites.

Speed Metric	Delta Speed (Approach to PC)	Standard Deviation	Delta Speed (Control Point to Midpoint)	Standard Deviation	N_1	N_2
Alabama Location #1 before	0.037	3.653	5.252	8.817	27	115
Alabama Location #1 1st after	-0.933	4.173	**0.549**	7.154	134	213
Alabama Location #1 2nd after	-0.685	3.947	***8.695***	10.378	73	226
Alabama Location #2 before	3.068	5.175	11.612	9.533	74	85
Alabama Location #2 1st after	3.127	3.909	**9.533**	7.555	220	246
Alabama Location #2 2nd after	2.526	3.984	10.210	7.864	234	276
Alabama Location #3 before	8.796	4.115	24.444	13.239	142	144
Alabama Location #3 1st after	**6.418**	3.244	**30.227**	13.700	225	234
Alabama Location #3 2nd after	***7.160***	3.253	***27.872***	12.219	306	298
Alabama Location #4 before	7.253	4.303	16.891	11.210	99	101
Alabama Location #4 1st after	**8.534**	3.981	16.823	9.590	236	237
Alabama Location #4 2nd after	**8.560**	4.400	***14.489***	10.179	241	229
Alabama Location #5 before	0.541	5.450	1.820	8.132	37	111
Alabama Location #5 1st after	0.097	4.304	**-3.264**	6.864	165	174
Alabama Location #5 2nd after	-0.210	3.920	***-1.738***	6.337	176	191
Alabama Location #6 before	-0.611	3.568	-0.964	12.250	54	56
Alabama Location #6 1st after	**7.915**	4.975	0.732	9.227	141	153
Alabama Location #6 2nd after	***0.327***	4.516	1.197	10.233	49	228
Alabama Location #7 before	6.827	3.501	14.381	8.213	127	126
Alabama Location #7 1st after	**4.925**	2.926	**16.709**	9.636	319	323
Alabama Location #7 2nd after	***6.089***	2.951	**15.883**	8.963	382	369
Alabama Location #8 before	3.838	4.295	0.290	6.497	105	100
Alabama Location #8 1st after	3.607	3.797	**2.405**	6.296	196	200
Alabama Location #8 2nd after	***4.479***	3.945	**2.308**	7.735	217	211

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
PC = Point of Curvature
N_1 = Number of observations approach to PC
N_2 = Number of observations control point to midpoint

As table 80 shows, there were many changes in the delta speed between the approach and the PC and the delta speed between the control point and the curve midpoint in Alabama. As table 80 shows, the speed difference from the approach to the PC increased for two sites in the first after period, while the difference also decreased for two sites. The difference increased by approximately 8.5 mph at Alabama Location #6. This was a result of vehicles having a higher speed at the approach in the first after period. There was also a significant increase in delta speed of approximately 1.3 mph at Alabama Location #6. This was a result of vehicles having a slower speed at the PC in the first after period. The difference decreased by approximately 2.4 mph at Alabama Location #3. This was a result of vehicles having a slower speed at the approach in the first after period. There was also a significant decrease of approximately 1.9 mph at Alabama Location #7. This was a result of vehicles having a slower speed at the approach in the first after period.

Most of the significant changes in delta speed (approach to PC) from the first after period were also significant in the second after period. The delta speed at Alabama Location #6 returned closer to the before period, and the difference was no longer significant in the second after period. The delta speed at Alabama Location #3 returned closer to the before period, but the difference was still significant, with a decrease of approximately 1.6 mph. The delta speed at Alabama Location #4 was similar to the first after period, with a delta increase of approximately 1.3 mph from the before period. The delta speed at Alabama Location #7 returned closer to the before period, but the difference was still significant, with an increase of approximately 0.7 mph.

As table 80 shows, the speed difference from the control point to the curve midpoint increased for three sites in the first after period, while the difference also decreased for three sites. The delta speed at Alabama Location #1 significantly decreased by approximately 4.7 mph. This was a result of vehicles having a slower speed at the control point. There was also a decrease in delta speed at Alabama Location #5 of approximately 5.1 mph. This was a result of speeds actually being higher at the curve midpoint than at the control point. Delta speed also decreased at Alabama Location #2 by approximately 2.1 mph. This was a result speeds at the curve midpoint increasing in the first after period.

In addition to the sites where there was a delta speed (control point to midpoint) difference from the before to first after period, there were also other sites where there was a change in delta speed in the second after period. Delta speed actually increased at Alabama Location #1 by approximately 3.4 mph, while there was a decrease in the first after period. This was a result of vehicles having a higher speed at the control point. The delta speed at Alabama Location #8 of was similar to the first after period, with a delta increase of approximately 2.0 mph from the before period. This was a result of vehicles having a higher speed at the control point and a lower speed at the curve midpoint. There was an increase in delta speed at Alabama Location #3 of approximately 3.4 mph. This was a result of vehicles having a higher speed at the control point. There was a decrease in delta speed at Alabama Location #4 of approximately 2.4 mph. This was a result of control point speeds being slightly lower. There was a decrease in delta speed at Alabama Location #5 of approximately 3.6 mph. Speeds actually increased from the control point to the curve midpoint. There was an increase in delta speed at Alabama Location #7 of approximately 1.5 mph. This was a result of vehicles having a slower speed at the curve midpoint.

There were many changes in delta speed, but there was no clear trend in the change. Delta speed increased at several sites, while it also decreased at other sites. Many of the speed changes were a result of control point speeds changing from the before to after periods. These changes at the control point make it difficult to determine whether the OSBs had an effect on delta speed. The change in delta speed varied both in direction and magnitude across the different Alabama sites, with no strong trend present.

Massachusetts

Table 81 shows the change in speed (delta) from the approach to the PC and from the control point to the curve midpoint.

Table 81. Change in speeds for Massachusetts sites.

Speed Metric	Delta Speed (Approach to PC)	Standard Deviation	Delta Speed (Control Point to Midpoint)	Standard Deviation	N_1	N_2
NB Braley Hill Road before	0.714	2.685	5.769	5.783	91	91
NB Braley Hill Road after	**-1.313**	3.009	**-2.042**	5.485	80	96
NB New Boston Road before	-1.409	3.472	4.805	6.582	44	77
NB New Boston Road after	-0.551	3.731	3.373	9.249	49	102
NB Reed Road before	0.380	2.432	11.155	7.481	50	110
NB Tucker Road before	3.889	2.603	-1.931	5.164	72	58
NB Tucker Road after	**1.027**	3.820	**0.293**	6.415	75	75
SB Braley Hill Road before	4.021	4.786	2.402	7.960	94	82
SB Braley Hill Road after	**11.126**	5.021	**5.765**	7.726	103	102
SB Tucker Road before	0.957	3.440	9.717	7.580	117	120
SB Tucker Road after	0.976	5.015	-	-	42	-
SB Reed Road before	1.563	3.956	0.343	5.573	64	73
SB Reed Road after	-	-	**3.566**	6.116	-	53
SB New Boston Road before	0.125	3.485	-0.167	7.053	64	66
SB New Boston Road after	**-0.809**	3.465	0.1	6.208	110	120

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
NB = Northbound
SB = Southbound
PC = Point of Curvature
N_1 = Number of observations approach to PC
N_2 = Number of observations control point to midpoint

As table 81 shows, there were many changes in the delta speed between the approach and the PC and the delta speed between the control point and the curve midpoint in Massachusetts. Delta speed (approach to PC) on northbound Braley Hill Road decreased by approximately 2.0 mph. In the before period delta speed was positive, which represents a decrease in speed from the approach to the PC. However, in the after period delta speed (approach to PC) was negative. Delta speed (control point to midpoint) also decreased in the after period on northbound Braley Hill Road by approximately 7.8 mph. Delta speed (control point to midpoint) was also negative in the second after period, which was a result from the control point speed being significantly lower and vehicles maintaining a more constant speed throughout the curve. As previously discussed, this was most likely a result of measurement error with the on-pavement sensor at the control point. There was also a decrease in delta speed (approach to PC) on northbound Tucker Road of approximately 2.9 mph. However, delta speed (control point to midpoint) increased in the first after period by approximately 2.2 mph. Delta speed was negative in the before period, but was positive in the first after period. Next, delta speed (approach to PC) and delta speed (control point to midpoint) increased by approximately 7.1 and 3.4 mph, respectively, on southbound Braley Hill Road. The large increase in delta speed (approach to PC) was the result of a significant increase in speeds at the approach, which was most likely the result of a measurement error with the on-pavement sensor, as previously discussed. Delta speed (control point to midpoint) increased as a result of speeds being lower at the curve midpoint in the first after period. Next, there was an increase in delta speed (control point to midpoint) on southbound Reed Road of approximately 3.2 mph. This was a result of lower speeds at the curve midpoint in the after period. Because approach speeds were not collected at this site in the first after period, a delta speed between the approach and PC was not calculated. Finally, delta speed (approach to PC) on southbound New Boston Road decreased by approximately 0.9 mph. This was a minimal decrease, and vehicles essentially maintained their speed from the approach to PC in both the before and first after period.

Unlike the Arizona sites, there was a weak trend in the change of delta speed at the Massachusetts sites. If the change in delta speed between the approach and PC on southbound Braley Hill Road was ignored because there most likely was a problem with the on-pavement sensor, the delta speed between the approach and PC decreased at three sites and remained the same at two others. If the change in delta speed between the control point and curve midpoint on southbound Braley Hill Road was ignored because there most likely was a problem with the on-pavement sensor, the delta speed between the control point and curve midpoint decreased at two sites, increased at one site, and remained the same at two sites. The OSBs may have a small effect on the delta speed between the approach and the PC and the delta speed between the control point and curve midpoint by decreasing the difference slightly.

Analysis of Variance for Speed Differences

An analysis of variance was used to compare speed differences in the before and after periods. The results are organized and grouped by State.

Arizona

As table 82 shows, the speed difference between the control point and the curve midpoint in the first after period significantly increased by 2.82 mph, which was a result of speeds at the curve midpoint decreasing and speeds at the control point increasing. The speed difference also increased in the second after period; however, this increase was not statistically significant. After installing the additional OSBs, the speed difference did not decrease from the control point to curve midpoint after the novelty effect period.

Table 82. Speed difference between the control point and the curve midpoint.

Time Period	*F* Statistic	Significance	Magnitude of Difference (mph)
First after period	35.62	**< 0.001**	**2.82**
Second after period	1.35	0.246	0.57
Second after period (additional OSBs installed)	2.71	0.100	-0.92

Bold indicates significance at the 95-percent confidence level (α=0.05)
OSB = Optical Speed Bar

As table 83 shows, there was an increase in the speed difference from the approach to the PC in the first after period; however, this increase was not significant. There was a significant increase in the speed difference in the second after period and after installing the additional OSBs of

1.33 and 1.41 mph, respectively. The increase in the difference from the approach to the PC in the second after period may show that the OSBs were effective in reducing drivers' speeds while in the OSBs.

Table 83. Speed difference between the approach and the PC.

Time Period	*F* Statistic	Significance	Magnitude of Difference (mph)
First after period	8.39	0.004	0.73
Second after period	26.11	**<0.001**	**1.33**
Second after period (additional OSBs installed)	23.08	**<0.001**	**1.41**

Bold indicates significance at the 95-percent confidence level (α=0.05)
OSB = Optical Speed Bar

As table 84 shows, the speed difference from the control point to the PC in the first before period significantly increased by 2.20 mph. The speed difference also increased in the second after period and after installing the additional OSBs; however, these increases were not significant.

Table 84. Speed difference between the control point and the PC.

Time Period	*F* Statistic	Significance	Magnitude of Difference (mph)
First after period	26.00	**<0.001**	**2.20**
Second after period	1.11	0.292	0.47
Second after period (additional OSBs installed)	0.35	0.552	0.30

Bold indicates significance at the 95-percent confidence level (α=0.05)

Alabama

As table 85 shows, the speed difference from the control point to the curve midpoint in the first after period did not significantly change. The speed difference significantly increased in the second after period by approximately 0.68 mph. After installing the additional OSBs, the speed difference decreased slightly from the control point to the curve midpoint. This decrease was not practically significant.

Table 85. Speed difference between the control point and the curve midpoint.

Time Period	*F* Statistic	Significance	Magnitude of Difference (mph)
First after period	2.14	0.143	0.51
Second after period	4.38	**0.036**	**0.68**

Bold indicates significance at 95 percent confidence level (α=0.05)

As table 86 shows, the speed difference from the approach to the PC significantly decreased in the after periods, which means that vehicles maintained a more constant speed as they approached the curve.

Table 86. Speed difference between the approach and the PC.

Time Period	*F* Statistic	Significance	Magnitude of Difference (mph)
First after period	34.65	**<0.001**	**-0.81**
Second after period	28.68	**<0.001**	**-0.7**

Bold indicates significance at the 95-percent confidence level (α=0.05)

As table 87 shows, the speed difference from the control point to the PC significantly decreased in the first after period, but there was no significant change in the second after period. The decrease in the first after period was approximately 1.68 mph. This corresponds with the decrease in the difference from the approach to the PC, and vehicles maintained a more constant speed from the control point to the PC.

Table 87. Speed difference control Point to PC.

Time Period	*F* Statistic	Significance	Magnitude of Difference (mph)
First after period	20.93	**<0.001**	**-1.68**
Second after period	0.05	0.829	0.08

Bold indicates significance at the 95-percent confidence level (α=0.05)

Massachusetts

As table 88, table 89, and table 90 show, all three speed differences decreased in the after period at the Massachusetts sites, with speed becoming more uniform throughout the horizontal curves. There was much variability in the speed changes at the different sites in Massachusetts, with the speed differences increasing at some sites, while decreasing at others. Overall, the analysis of variance shows the speed difference decreased from the control point to the curve midpoint, the approach to the PC, and the control point to the PC, with speed difference decreases of 1.80, 1.88, and 4.87 mph, respectively.

Table 88. Difference between the control point and the curve midpoint.

Time Period	*F* Statistic	Significance	Magnitude of Difference (mph)
After period	48.78	**<0.001**	**-1.80**

Bold indicates significance at the 95-percent confidence level (α=0.05)

Table 89. Difference between the approach and the PC.

Time Period	*F* Statistic	Significance	Magnitude of Difference (mph)
After period	71.87	**<0.001**	**-1.88**

Bold indicates significance at the 95-percent confidence level (α=0.05)

Table 90. Difference between the control point and the PC.

Time Period	*F* Statistic	Significance	Magnitude of Difference (mph)
After period	508.45	**<0.001**	**-4.87**

Bold indicates significance at the 95-percent confidence level (α=0.05)

Vehicles Exceeding the Posted Speed Limit

Finally, in addition to the *t*-test, the percentage of vehicles exceeding the PSL and the advisory speed at the treatment and control sites was calculated and compared between data-collection periods. The following sections present the results for each State.

Arizona

Table 91 shows the number of passenger cars observed as exceeding the speed limit at each location at every site in Arizona in the before and both after periods. A test of proportions was used to determine whether the proportion of speeding vehicles changed at each location of every site from the before to after treatment application. From table 91, it was clear there were no visible trends from the before period to after treatment application. There was no change in the proportion of passenger cars exceeding the speed limit at the control point at any site, in any analysis period. The only change in proportion of passenger cars exceeding the speed limit at the approach was in the second after period on southbound Diamond Bar Road, where slightly fewer vehicles exceeded the speed limit. These results were expected because these data collection points were before or at the beginning of the OSBs, so no change would be expected. The only change at the PC between the before and after periods was the proportion of vehicles exceeding the PSL on northbound Pierce Ferry Road decreased in the second after period. The effect of the OSBs on the proportion of vehicles exceeding the speed limit varies from site to site and from the first to second after period. On northbound Pierce Ferry Road, the proportion decreased in the first after period, but then it increased in the second after period. On southbound County Route 1, the proportion decreased in the first after period, but then returned to the proportion in the before period. On southbound Diamond Bar Road, the proportion increased in the first after period, but then decreased in the second after period. On southbound Shinarump Road, the proportion decreased in the second after period. The fact that there was no trend in the change of the proportion of vehicles exceeding the speed limit and those with decreased speeds, while others increased, probably means the OSBs had no effect.

Table 91. Number of vehicles exceeding the PSL in Arizona.

Location and Data Collection Period		Speed Observation Site: NB Pierce Ferry Rd	SB County Route 1	SB Diamond Bar Rd	SB Shinarump Rd
Control Point PCs	Before	83	145	115	55
	1st after	138	173	73	62
	2nd after	127	100	89	90
Approach PCs	Before	69	147	95	71
	1st after	106	174	63	70
	2nd after	101	102	***96***	101
PC PCs	Before	57	139	80	59
	1st after	60	166	25	52
	2nd after	***39***	97	*47*	61
Midpoint PCs	Before	21	122	80	55
	1st after	**15**	**135**	**43**	56
	2nd after	*77*	***58***	***30***	***62***
Number of PCs at Control Point and Approach	Before	108	147	136	96
	1st after	186	175	92	98
	2nd after	162	102	108	146
Number of PCs at PC	Before	90	141	99	90
	1st after	102	169	37	93
	2nd after	161	97	53	101
Number of PCs at Midpoint	Before	107	129	99	73
	1st after	180	154	64	72
	2nd after	148	60	105	136

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
NB = Northbound
SB = Southbound
PC = Point of Curvature

Alabama

Table 92 shows the number of passenger cars observed to exceed the speed limit at each location at every site in Alabama in the before and both after periods. A test of proportions was used to determine whether the proportion of speeding vehicles changed at each location of every site from the before to after treatment application. From table 92, it was clear there were no long-term effects on the number of vehicles speeding after the application of the OSBs.

Alabama locations #1, #2, and #3 were the only sites where there was a change in the proportion of vehicles exceeding the speed limit at the control point. The proportion decreased at Alabama Location #1 in the first after period, but returned to the before period proportion in the second after period. The proportion decreased at Alabama Location #2 in the second after period, but increased at Alabama Location #3. The proportion of vehicles exceeding the speed limit at the approach decreased in the first after period at Alabama locations #1, #3, and #7, but the decreased proportions were only maintained through the second after period at Alabama Location #3. The proportion of vehicles exceeding the speed limit at the approach increased for Alabama Location #6 in the first after period and Alabama Location #8 in the second after period. The only change in the proportion of vehicles exceeding the speed limit at the PC was a decrease as in the first after period. The proportion of vehicles exceeding the speed limit decreased during the first after period at the following locations: #6, #8, #4, #2, and #7. However, the reductions were only maintained through the second after period at locations #6 and #7. The proportion of vehicles exceeding the speed limit increased at Location #5 during both after periods. There was no visible long-term trend in the effects of the OSBs on the proportion of vehicles exceeding the PSL. There was a decrease at the curve midpoint for only two sites in the second after period, but there was also an increase at another site. The fact that there was no consistent trend in the change of the proportion of vehicles exceeding the speed limit, but no change at most sites, probably means the OSBs had no effect.

Table 92. Number of vehicles exceeding the PSL in Alabama.

Location and Data Collection Period		Speed Observation Site							
		Alabama Location #6	**Alabama Location #1**	**Alabama Location #8**	**Alabama Location #3**	**Alabama Location #4**	**Alabama Location #5**	**Alabama Location #2**	**Alabama Location #7**
Control Point PCs	Before	85	28	107	131	100	75	22	126
	1st after	109	**5**	171	214	226	135	40	314
	2nd after	180	***50***	***216***	**302**	239	***177***	**43**	383
Approach PCs	Before	96	24	109	31	99	90	2	122
	1st after	**156**	**14**	193	**19**	231	165	7	**284**
	2nd after	***196***	26	**229**	**42**	238	192	7	***364***
PC PCs	Before	47	9	92	0	86	30	0	89
	1st after	110	**8**	174	1	**157**	117	2	197
	2nd after	38	***13***	193	4	***199***	125	3	236
Midpoint PCs	Before	42	5	92	1	44	50	2	105
	1st after	**79**	3	**152**	0	**62**	**128**	**0**	**168**
	2nd after	**119**	7	***178***	***7***	***108***	**143**	1	***230***
Number of PCs at Control Point & Approach	Before	118	125	117	152	103	118	97	128
	1st after	165	218	201	235	237	244	250	323
	2nd after	253	256	234	323	244	277	303	393
Number of PCs at PC	Before	54	27	105	142	99	37	74	127
	1st after	141	134	196	225	236	165	220	319
	2nd after	49	73	217	306	241	176	303	382
Number of PCs at Midpoint	Before	56	115	100	144	101	111	85	126
	1st after	153	213	200	234	237	174	246	323
	2nd after	228	226	211	298	229	191	234	369

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period

Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods

PC = Point of Curvature

Massachusetts

Table 93 shows the number of passenger cars observed that exceeded the speed limit at each location at every site in Massachusetts in the before and both after periods. A test of proportions was used to determine whether the proportion of speeding vehicles changed at each location of every site from the before to after treatment application. From table 93, it was clear there were no visible trends from the before period to after treatment application. Northbound Braley Hill Road and northbound Tucker Road were the only sites where there was a change in the proportion of vehicles exceeding the speed limit at the control point. The proportion decreased on northbound Braley Hill Road, but increased on northbound Tucker Road. Northbound Tucker Road was the only site where there was a change in the proportion of vehicles exceeding the speed limit at the approach. The proportion increased, which corresponds with the increase in the proportion at the control point. Northbound Braley Hill Road and northbound Tucker Road were the only sites where there was a change in the proportion of vehicles exceeding the speed limit at the PC. The proportion increased at both sites, which corresponds with the increase in the proportion at the control point of northbound Tucker Road. The increase on northbound Braley Hill Road does not correspond with the change at the control point because the proportion decreased there. Finally, these two sites were the only sites where there was a change in the proportion of vehicles exceeding the speed limit at the curve midpoint. The proportion increased at both sites, which corresponds with the increase in the proportion at the control point of northbound Tucker Road. Again, the increase does not correspond with the change at the control point, because the proportion decreased there. There was no visible trend in the effects of the OSBs on the proportion of vehicles exceeding the PSL. There was a change at two sites only, and at one site, there was also an increase in the proportion at the control point. The fact that there was no trend in the change of the proportion of vehicles exceeding the speed limit, with an increase at several sites but no change at most sites, probably means the OSBs had no effect.

Table 93. Number of vehicles exceeding the PSL in Massachusetts.

Location and Data Collection Period		Speed Observation Site							
		NB Braley Hill Rd	NB New Boston Rd	NB Reed Rd	NB Tucker Rd	SB Braley Hill Rd	SB Tucker Rd	SB Reed Rd	SB New Boston Rd
Control Point PCs	Before	69	69	116	27	95	124	102	49
	1st after	**33**	116	-	**68**	106	110	53	110
	2nd after	0	0	0	0	0	0	0	0
Approach PCs	Before	28	54	114	**51**	95	116	101	56
	1st after	39	97	-	77	109	108	-	104
	2nd after	0	0	0	0	0	0	0	0
PC PCs	Before	18	38	50	27	79	103	64	43
	1st after	**40**	37	-	**72**	90	38	49	85
	2nd after	0	0	0	0	0	0	0	0
Midpoint PCs	Before	28	38	72	32	69	106	72	42
	1st after	**45**	64	-	**58**	77	-	51	85
	2nd after	0	0	0	0	0	0	0	0
Number of PCs at Control Point & Approach	Before	91	85	116	73	98	127	103	75
	1st after	100	143	-	81	109	114	53	148
	2nd after	0	0	0	0	0	0	0	0
Number of PCs at PC	Before	91	44	50	72	94	117	64	64
	1st after	80	49	-	75	103	42	51	110
	2nd after	0	0	0	0	0	0	0	0
Number of PCs at Midpoint	Before	91	77	110	58	82	120	73	66
	1st after	96	102	-	75	102	-	53	120
	2nd after	0	0	0	0	0	0	0	0

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
NB = Northbound
SB = Southbound
PC = Point of Curvature

Speed Variance Analysis

The final speed performance metric considered in the current study was speed variance. A two-sided *F*-test was used to compare the variances of vehicle operating speeds in the before and after periods for passenger cars and heavy trucks. The results are organized by State.

Arizona

Table 94 shows the SD of speeds at each location at every site in Arizona for the before and both after periods. From table 93, it was clear there were no visible trends in a change in SD from the before period to first after period. The SD of control point speeds did not change at any site in the first after period. Northbound Pierce Ferry Road was the only site where there was a change in the SD of approach speeds, which was an increase of approximately 1.3 mph. There was also an increase of approximately 1.2 mph in the SD of speeds at the PC of northbound Pierce Ferry Road. There was no change in the SD of speeds at the PC at any other site. Finally, there was no change in the SD of speeds at the curve midpoint of any site.

As in the first after period, there were no visible trends in the change in SD from the before to second after period. The only change at the control point was on southbound Shinarump Road, where the SD increased by approximately 1.3 mph. There was no change in the SD of the speeds at the approach. There was a change in the SD of speeds at the PC for both northbound Pierce Ferry Road and southbound Shinarump Road. The SD decreased by approximately 1.0 mph on northbound Pierce Ferry Road and increased by approximately 3.3 mph on southbound Shinarump Road. The increase on southbound Shinarump Road was still significant even after accounting for the increase at the control point. Finally, there was no change in the SD of speeds at the curve midpoint between the second after period and the before period. If the OSBs had any effect on the speed deviation, it would have been most prevalent at the PC and curve midpoint, because these were after the OSBs. The fact that there was no consistent trend in the change of the SD of speeds at most sites probably means the OSBs had no effect.

Table 94. Change in standard deviation of speed in Arizona.

Site	Control Point SD	Approach SD	PC SD	Midpoint SD	N_1	N_2	N_3
NB Pierce Ferry Road before	7.308	7.121	5.795	6.695	108	90	107
NB Pierce Ferry Road 1st after	7.836	**8.415**	**6.970**	5.989	186	102	180
NB Pierce Ferry Road 2nd after	7.377	***6.373***	***4.753***	***7.012***	162	161	148
SB County Route 1 before	9.132	5.325	4.846	6.655	147	141	129
SB County Route 1st after	9.497	5.280	4.544	7.241	175	169	154
SB County Route 2nd after	9.920	5.562	5.347	7.531	102	97	60
SB Diamond Bar Road before	8.543	7.230	6.758	7.590	136	99	99
SB Diamond Bar 1st after	9.450	7.161	8.636	8.862	92	37	64
SB Diamond Bar 2nd after	9.244	8.364	6.781	7.215	108	53	105
SB Shinarump Road before	7.047	8.893	6.248	7.692	96	90	73
SB Shinarump Road 1st after	6.374	7.863	6.713	8.109	98	93	72
SB Shinarump Road 2nd after	***8.309***	***9.902***	***9.530***	6.813	146	101	136

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
NB = Northbound
SB = Southbound
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Alabama

Table 95 shows the SD of speeds at each location at every site in Alabama for the before and both after periods. From table 95, it was clear there were no visible trends in the change in SD from the before period to first after period. The only change in the SD at the control point was at Location #1, which was a decrease of approximately 1.8 mph. The OSBs were not visible at this location, which corresponds to the SDs remaining the same as the before period. The SD decreased at the approach at locations #8 and #4 by approximately 2.1 and 2.2 mph, respectively. The SD also decreased at the PC at Location #1 and Location #2 by approximately 3.4 and 1.5 mph, respectively. The SD decreased at the curve midpoint at Location #4 and Location #2 by approximately 2.0 and 2.4 mph, respectively. However, the SD increased at the curve midpoint at Location #3 by approximately 2.4 mph.

As in the first after period, there were no visible trends in the change in the SD from the before to second after period. The SD decreased at the control point at Location #4 by approximately 1.7 mph. As in the first after period, the SD decreased at the approach at Location #8 and Location #3 by approximately 1.9 and 1.5 mph, respectively. The SD also increased at the approach at Location #6 by approximately 1.3 mph. The only change at the PC was the SD increased at Location #3 by approximately 0.6 mph. The SD increased at the curve midpoint at Location #1 by approximately 2.4 mph. As in the first after period, the SD increased at the curve midpoint at Location #3 by approximately 1.3 mph. The SD decreased at the curve midpoint at Location #2 by approximately 1.8 mph. If the OSBs had any effect on the speed deviation, it would have been most prevalent at the PC and curve midpoint, because these were after the OSBs. The SD decreased at the PC for two sites in the first after period, but these decreases did not persist throughout the second after period. The fact that there was no consistent trend in the change of the SD of speeds at most sites, with an increase at some sites, probably means the OSBs had no effect.

Table 95. Change in standard deviation of speed in Alabama.

Site	Control Point SD	Approach SD	PC SD	Midpoint SD	N_1	N_2	N_3
Alabama Location #1 before	8.953	7.773	8.541	6.979	125	27	115
Alabama Location #11st after	**7.113**	6.823	**5.181**	6.102	218	134	213
Alabama Location #1 2nd after	***7.959***	6.901	***7.593***	***9.344***	256	73	226
Alabama Location #2 before	7.218	5.239	5.980	7.485	97	74	85
Alabama Location #2 1st after	6.986	4.882	**4.532**	**5.038**	250	220	246
Alabama Location #2 2nd after	7.132	5.105	***5.221***	***5.651***	303	234	276
Alabama Location #3 before	12.267	6.872	4.058	5.495	152	142	144
Alabama Location #3 1st after	11.686	**4.676**	4.349	**7.731**	235	225	234
Alabama Location #3 2nd after	11.413	***5.385***	**4.655**	***6.809***	323	306	298
Alabama Location #4 before	9.000	5.784	4.627	7.770	103	99	101
Alabama Location #4 1st after	8.434	5.462	4.126	**5.790**	237	236	237
Alabama Location #4 2nd after	***7.286***	5.883	***4.677***	***8.131***	244	241	229
Alabama Location #5 before	6.783	7.346	6.959	6.603	118	37	111
Alabama Location #5 1st after	6.687	6.808	6.126	7.155	244	165	174
Alabama Location #5 2nd after	6.704	7.059	6.159	7.098	277	176	191
Alabama Location #6 before	9.141	5.970	6.466	8.027	118	54	56
Alabama Location #6 1st after	8.521	**8.573**	6.578	7.108	165	141	153
Alabama Location #6 2nd after	***9.615***	***7.279***	7.619	7.857	253	49	228
Alabama Location #7 before	9.446	5.276	4.730	5.497	128	127	126
Alabama Location #7 1st after	9.866	5.229	4.307	5.052	323	319	323
Alabama Location #7 2nd after	9.392	5.449	4.246	5.431	393	382	369
Alabama Location #8 before	5.742	8.262	4.442	5.804	117	105	100
Alabama Location #8 1st after	6.192	**6.198**	4.723	5.335	201	196	200
Alabama Location #8 2nd after	6.249	**6.372**	4.895	***6.486***	234	217	211

Bold indicates significance at the 95-percent confidence level (α=0.05) between the 1st/2nd after period and before period
Italics indicates significance at the 95-percent confidence level (α=0.05) between the 1st and 2nd after periods
SD = Standard Deviation
PC = Point of Curvature
N_1 = Number of observations at the control point and approach
N_2 = Number of observations at the PC
N_3 = Number of observations at the midpoint

Massachusetts

Table 96 shows the SD of speeds at each location at every site in Massachusetts for the before and both after periods. Unlike at the Arizona sites, there may have been a change in the SD of speeds after installing the OSBs. The SD of speeds at the control point increased by approximately 1.6 mph on northbound New Boston Road, decreased by approximately 0.6 mph on northbound Tucker Road, and decreased by approximately 2.4 mph on southbound Tucker Road. These changes were not the result of applying the OSBs because they were not visible from the control point. The SD of speeds at the approach increased by approximately 1.0 mph on northbound Braley Hill Road, 1.3 mph on northbound Tucker Road, and 1.5 mph on southbound Braley Hill Road. The optical speeds bars may have affected these SDs as they would be visible prior to reaching the approach. The SD of speeds at the PC increased by approximately 1.1 mph on northbound Braley Hill Road, 4.1 mph at southbound Reed Road, and 0.8 mph on southbound New Boston Road. Finally, the SD of speeds at the curve midpoint increased by approximately 2.2 mph on northbound Tucker Road. All of these changes were still significant after accounting for the changes at the control point, because there was no change at the control point for any of these sites, except northbound Tucker Road, but the SD decreased at that site. The SD at the approach increased between 1.0 and 1.5 mph at three sites and remained the same at the other three in the after period. The SD at the PC increased between 0.8 and 4.1 mph at three sites and remained the same at the other four sites in the after period. The SD at the curve midpoint increased by 2.2 mph at one site and remained the same at the five other sites. It appears the SD may increase slightly at the approach and PC, while remaining the same at the curve midpoint after installing the OSBs.

Table 96. Change in standard deviation of speed in Massachusetts.

Site	Control Point SD	Approach SD	PC SD	Midpoint SD	N_1	N_2	N_3
NB Braley Hill Road before	5.855	3.912	3.202	3.698	91	91	91
NB Braley Hill Road 1st after	5.390	**4.908**	**4.269**	3.891	100	80	96
NB New Boston Road before	5.523	6.106	4.877	7.002	85	44	77
NB New Boston Road 1st after	**6.820**	6.336	5.826	8.009	143	49	102
NB Tucker Road before	4.471	3.708	3.388	4.059	73	72	58
NB Tucker Road 1st after	**3.653**	**5.005**	3.625	**6.242**	81	75	75
SB Braley Hill Road before	4.519	4.577	4.536	7.847	98	94	82
SB Braley Hill Road 1st after	4.856	**6.085**	3.957	7.277	109	103	102
SB Tucker Road before	7.357	4.281	4.870	4.579	127	117	120
SB Tucker Road 1st after	**4.964**	4.673	5.861	0.000	114	42	0
SB Reed Road before	5.203	5.096	4.094	5.093	103	64	73
SB Reed Road 1st after	4.587	0.000	**8.267**	4.757	53	51	53
SB New Boston Road before	6.132	5.332	4.496	5.412	75	64	66
SB New Boston Road 1st after	5.952	5.376	**5.277**	6.162	148	110	120

Bold indicates significance at the 95-percent confidence level (α=0.05)
NB = Northbound
SB = Southbound
SD = Standard Deviation
PC = Point of Curvature
N1 = Number of observations at the control point and approach
N2 = Number of observations at the PC
N3 = Number of observations at the midpoint

SUMMARY

Results of this study showed that OSBs may have some minor effects on vehicle speeds. However, the magnitude of speed reductions was generally small, and because of the inconsistent magnitude of speed reductions at all the test sites, no conclusion can be drawn regarding the effect of OSB treatment to reduce speeds. Even though minor speed reductions occurred at some locations, average and 85th percentile speeds observed at all the sites were still higher than the PSLs, indicating the OSBs were not effective enough in providing the desired speed limit compliance. Future analyses may consider featuring an enforcement component, which was out of the scope of this study.

It is clear from the data that driver operating speeds are influenced by the curve radius. Further, the benefits gained by a reduction in driver operating speeds before the PC may be lost as drivers compensate and accelerate through the curve (for treatments that end prior to the PC). While this study did not show any consistent benefits as a result of installing the OSBs, consideration should be given to extending the treatment through the curve—keeping in mind that the entrance and midpoint are the most severe areas.

6. SAFETY EFFECTS OF LANE-WIDTH–SHOULDER-WIDTH COMBINATIONS ON RURAL, TWO-LANE ROADS

This section of the report describes a study to estimate the safety effects of lane width–shoulder width combinations on rural two-lane, two-way roads. This report first describes the background and study motivation. Next, it discusses the process for collecting safety data, including steps for identifying and defining segments, measuring geometric features, and gathering traffic and crash data. The research team then presents the statistical analysis methods for estimating the effects of lane and shoulder width allocation on both crash frequencies and severities; this is followed by the model estimation results and interpretation. The section concludes with a discussion of major findings and considerations for future research.

OVERVIEW

The conclusion that safety increases as lane width increases is based on the premise that wider lanes reduce the consequences of driver deviations from their intended path. Vehicles traveling in opposite directions on undivided, two-way roads are separated by larger distances if lanes are wider. Vehicles traveling in the same direction on multilane roads are also separated by larger distances if lanes are wider. Wider lanes provide more room for recovery in near-crash situations (e.g., evasive maneuvers to miss an object on the roadway or inadvertently drifting toward the roadside).

Using some of the same logic, it is also assumed that wider shoulders are associated with increased safety because they provide increased separation between a parked or disabled vehicle and the traveled way. Wider shoulders also provide more room for recovery following an unintentional (e.g., inattentive driver drift) or intentional (e.g., evasive maneuver) shoulder encroachment. Wider shoulders are also associated with larger lateral clearances from roadside objects and longer available sight distances when there are sight obstructions to the inside of horizontal curves.

Wider lanes and shoulders also appear to result in faster operating speeds.[47] For example, methodologies in the *Highway Capacity Manual* (HCM) predict an approximately 1.3 to 1.7 mph increase in speeds for every 1 ft increase in shoulder width on two-lane highways. Similarly, the HCM 2010 predicts an approximately 0.4 to 1.1 mph increase in speeds on two-lane highways for every 1 ft increase in lane width. [37] All else being equal, drivers traveling at faster speeds may be less likely to successfully react to unexpected situations (e.g., changes in lead vehicle behavior, non-motorized users crossing traveled way, changes in roadway alignment, roadside encroachments) than drivers traveling at slower speeds. In addition, the energy dissipated during a crash is directly proportional to the square of travel speed at the time of the crash. Impact forces that drivers experience increase as this initial speed increases and decreases as the time over which energy is dissipated decreases. The crash severity (i.e., probability of fatality or severe injury) therefore increases as initial travel speed increases.

The tradeoffs among speed, safety, lane width, and shoulder width are complex. Conclusions inferred from existing research are that wider cross sections tend to improve safety performance (i.e., fewer expected crashes); however, narrower cross sections tend to produce lower vehicle operating speeds. If these two points are independently true, then implementing reduced lane widths and shoulder widths as a speed management technique—a topic of interest for quite some time—should increase the number of crashes. However, some published safety studies indicate that there are cases where narrower cross sections are not necessarily associated with a higher expected crash frequency. For example, Gross et al. developed CMFs for lane-width–shoulder-width combinations on rural, two-lane highways.[48] The CMF estimation of interactions between lane width and shoulder width used a case-control methodology. Results yielded the following logical conclusions:

- Shoulder width has a larger effect on safety when lanes are narrow, but the effect of shoulder width decreases as lane width increases.
- An increase in lane width does not always increase safety, particularly when shoulder widths are wider.

Rural, two-lane highway segments with lane-width–shoulder-width combinations totaling 16 to 17 ft (e.g., 10-ft lanes with 6-ft shoulders; 11-ft lanes with 6-ft shoulders, 12-ft lanes with 5-ft shoulders) may not be any less safe than rural, two-lane highway segments meeting HSM base conditions (i.e., 12-ft lanes with 6-ft shoulders).

Bonneson and Pratt also estimated interactions between lane and shoulder width and drew similar conclusions—the safety effect of lane width depends on the shoulder width.[49] Research on lane width and safety on urban and suburban arterials, conducted as part of developing the HSM, concluded, "No consistent relationship was found between lane width and safety. Therefore, lane width was not included in the model."[78] Other research on lane width in urban areas indicated that the "Broad lane indicator (lanes wider than 12 ft) was associated with a higher frequency of urban section run-off-road accidents. A plausible explanation is that a broader lane could be expected to allow a higher traveling speed and thereby create a greater likelihood for run-off-road [crashes] on urban sections".[79]

The above-mentioned studies do have some limitations. For example, in the study by Gross et al. found the following:[48]

- Transferability of results across States was uncertain.
- Horizontal alignment was not considered.
- Several of the differences in cross-section combinations were not significant because of the limited sample sizes.

Harwood et al. did not report any model parameters for lane width, only the general conclusion about inconsistent relationships cited above.[78] No conclusive statements can be made to date other than considering lane width and shoulder width interactions appears to uncover cross-section combinations that will result in lower speeds (according to published speed prediction models) without a relative reduction in safety on rural two-lane roads. This is different than the current state-of-knowledge in the HSM and has the potential to contribute significantly to knowledge on geometric design, speed, and safety relationships and on the use of road geometry as a speed management technique.

Following a review of the literature conducted during an earlier task of this research, there was enough evidence for FHWA's technical advisory team for this project to support exploring different combinations of lane and shoulder widths as a strategy to reduce operating speeds and improve safety on rural roads. This strategy has the potential to be low cost by reallocating existing pavement widths. This study focuses only on safety effects of lane-width–shoulder-width combinations. The study addresses the following limitations to previously published speed and safety research.

Most of the speed- and safety-related geometric design research reviewed in earlier tasks of this research focused on the effects of design elements in isolation and then combined the isolated effects through some additive or multiplicative process. This research looks at lane-width–shoulder-width combinations as opposed to isolated effects and used a variety of variable specifications for lane and shoulder widths, including indicator variables, continuous variables, and interaction terms.

Speed and safety prediction models reviewed in earlier tasks of this research were estimated by relating crashes and speed at some location to traffic, roadway, and other surrounding features at that same location (e.g., a short, homogenous segment). Proponents of self-enforcing, self-explaining roads have argued that the using cross-sectional geometry as a speed management technique will appear effective only when applied and studied over a significant stretch of road.[(80)] This research looks at the safety effects of lane widths and shoulder widths over significant stretches of road as opposed to short, homogenous segments. The research team used alignment indices to capture changes in alignment over these road segments.

Databases used to estimate models were verified and enhanced using satellite imagery from Google Earth™. The research team checked and confirmed or (when necessary) recoded with the correct measurement to verify accuracy of every attribute (e.g., lane width, shoulder width) in the electronically coded databases used for model estimation. In addition, the existing electronically coded databases were enriched by adding new attributes (e.g., curve radii; driveway density; barrier presence, length, and offset; roadside hazard rating; pavement markings) to lower the chances of omitted variable bias.

The research team explored the effects of lane-width–shoulder-width combinations on crash severity by estimating severity distribution functions (SDF) in addition to estimating different crash frequency models for various severity levels. The databases used to estimate the severity models contain the same crashes and road segments as the frequency model databases, but were restructured so that the basic observation unit (i.e., database row) is the crash instead of the road segment. The severity models can be used to estimate the probability, or proportion, of each severity level given the traffic, geometric, and traffic control characteristics. The newest HSM edition proposes methodologies for SDFs on freeways and interchanges, which appears to be a promising approach; however, to date, the applications in applied safety analysis are relatively limited.

DATA COLLECTION

The data collection effort focused on rural two-lane, two-way road segments in Minnesota and Illinois. The study did not include safety relationships at intersections or on intersection approaches. Rural areas were defined as those outside of urban boundaries, but not suburban. Suburban areas may be inside or outside urban boundaries. Some subjectivity was involved in defining an area as suburban.

The research initially identified three Highway Safety Information System (HSIS) States as possible data alternatives for this study: Illinois, Minnesota, and North Carolina. These three States were initially selected because of the availability of a number of precipitating event variables that could provide several alternatives for identifying a "speed-related crash" and data for roads classified "lower than" arterials (i.e., collectors and local roads). However, a majority of rural roads in North Carolina have unpaved shoulders, which would limit the ability to study different pavement allocations between lanes and shoulders. Thus, the research team considered only data from Minnesota and Illinois for the analysis. Initially, having access to multiple alternatives for identifying a "speed-related crash" was an important criterion because the safety results were to be linked with observational speed studies conducted along the same segments in the selected States. However, FHWA decided to focus on a more general safety study of lane-width–shoulder-width combinations and spend the remaining resources on more extensive field data collection for other treatments studied as part of this research effort (OSBs and high-friction surfaces).

The safety effects of lane-width–shoulder-width combinations were estimated through a cross-sectional study design for two reasons: 1) State and local transportation agencies do not routinely reallocate existing pavement widths, particularly in rural areas; and 2) the proportion of all possible lane-width–shoulder-width combinations that could be evaluated using a before–after study during this project, and their respective sample sizes, would be limited.

To overcome some of the documented limitations of cross-sectional study designs, such as those outlined in FHWA's *A Guide to Developing Quality Crash Modification Factors* (FHWA-SA-10-32), the research team executed an in-depth, manual data collection effort to verify and enrich typical electronic databases often used for safety studies.[81] The research team also used modeling alternatives that address possible confounding factors between road segments with different lane-width–shoulder-width combinations not captured in the models (e.g., terrain, weather, driver characteristics, crash reporting thresholds, and practices). The following sections discuss these strategies.

DATA COLLECTION SOURCES, TOOLS, AND STRATEGIES

Data for this study included geometric features, traffic data, and crash data from rural roads. The research team used data collected from the following tools and information sources:

- HSIS: Roadway, traffic, and crash data were collected from FHWA's HSIS database.
- Geographic Information System (GIS) roadway databases from the Illinois and Minnesota Departments of Transportation: Roadway networks in GIS data formats are publicly available for download. The research team used ESRI ArcMap™ GIS software package to process the GIS files.

- Google Earth™ and Google Street View™: The research team used these two publicly available mapping and imaging tools to collect, check, and confirm the geometric features as well as enhance the electronic data with additional variables.

Database Verification and Enrichment

Safety models are commonly developed using electronically coded traffic and roadway data for independent variables; however, information in these datasets can be limited and inaccurate. Limited data means that the database may not include variables that could influence safety. Estimating safety models without these variables can result in omitted variable bias (i.e., erroneous parameter estimates for variables in the models). Including inaccurate information (e.g., incorrect dimensions for lane width) will result in biased parameter estimates. Based on this discussion, the research team applied the following techniques:

- Used electronically coded databases to verify the accuracy of every attribute used for estimating the safety models.

- Added new attributes to enrich existing databases.

The research team used the aforesaid tools and techniques to conduct verification and enhancement activities.

Geometric Features

The team used a combination of HSIS roadway data files, GIS roadway databases, and Google Earth™/Google Street View™ to collect roadway geometric characteristics. The team also manually defined each study segment. The segments were homogenous in terms of the number of lanes, lane width, shoulder width, shoulder type, and traffic volume. The segments did not include unsignalized or signalized intersections, and they began and ended at least 250 ft from adjacent intersections. The 250-ft number is consistent with research used to develop HSM predictive methods.[82]

The team explored different alignment indices to capture changes in horizontal alignment. This approach allowed longer segments to be defined in the database than those that would result from separating curves from tangents. Defining longer road segments was seen as a crucial step to determining whether geometry can be used to achieve desirable speed and safety effects. Most existing speed and safety prediction models relate crashes and speed at a specific location (e.g., a short, homogenous section) to traffic, roadway, and other surrounding features at that same specific location. The concept of driver expectancy recognizes that drivers make decisions based on previous experiences. Similarly, proponents of self-enforcing, self-explaining roads have argued that using geometry as a speed management technique appears effective only when applied and studied over a significant stretch of road.[80] Research to date has not captured this effect, but instead indicated that geometric design decisions made to reduce speeds (e.g., increased curvature, narrower lane widths) are expected to increase the expected number crashes.

The team defined road segments, collected their features and then checked and confirmed using Google Earth™'s satellite images and Google Street View™ photographs. The team used the

Ruler tools in Google Earth™ to measure distances and to check and confirm lane width, shoulder width, segment length, and barrier presence and length. These tools were also used to measure deflection angles and horizontal curve lengths. The research team developed a set of Google Earth™ calculation tools to automate this process. NCHRP 17-45, *Enhanced Safety Prediction Methodology and Analysis Tool for Freeways and Interchanges,* used similar techniques.(83) The team also developed new tools compatible with the rural, two-lane context of interest for this current FHWA effort.

Segment Location

The first step of the data-collection process was locating the roadway segment of interest in Google Earth™ and determining its mileposts. Google Earth™ and other publicly available mapping and satellite imaging tools do not provide milepost information. However, the milepost of any point along a given State route in Minnesota and Illinois can be determined by using a combination of HSIS roadway data, GIS data for roadway networks, and Google Earth™. The following discussion provides a step-by-step description of the process. Any point along a route was identified by all three of the following identifiers:

- County number and name (County).
- State route number (rte_nbr or s_rtenbr).
- Milepost.

HSIS roadway data files include all of these variables for both Illinois and Minnesota. The goal of this process was to then transfer these pieces of information from the HSIS roadway data files into Google Earth™ so the team could verify and enhance the data. HSIS data identified roadway segments by county, route number, and milepost. Google Earth™ uses coordinates. The team therefore used GIS data and ArcGIS™ software to translate location information from HSIS roadway inventory files into Google Earth™.

GIS roadway files from Illinois and Minnesota have different structures and information; therefore, the team could not use a single method of transferring HSIS data points into Google Earth™ through GIS for both States. Although there are many similarities, the procedure is described separately for Minnesota and Illinois. The procedure is illustrated by two examples, one for each State.

Minnesota Step 1: Collect route number, county number, and milepost from the HSIS roadway data file

Figure 120 shows an example of the segment location information in the HSIS roadway data file for Minnesota. In this example, the beginning point of the segment shown in the seventh row is the point of interest. It is used throughout the process of locating this segment in Google Earth™. The point is located on Route 1, in County 4, at milepost 110.453.

rte_sys	rte_nbr	begmp	endmp	county	seg_lng	no_lanes	year	lanewid	rodwycls
3	1	73.384	79.4	4	6.016	2	2007	12	8
3	1	79.4	85.395	4	5.995	2	2007	12	8
3	1	85.395	89.364	4	3.969	2	2007	12	8
3	1	89.364	90.357	4	0.993	2	2007	12	8
3	1	90.357	96.746	4	6.389	2	2007	12	8
3	1	110.124	110.453	4	0.329	2	2007	12	8
3	1	110.453	112.762	4	2.309	2	2007	12	8
3	1	112.762	112.817	4	0.055	2	2007	12	8
3	1	112.817	112.857	4	0.04	2	2007	12	8
3	1	112.857	112.927	4	0.07	2	2007	12	8
3	1	112.927	112.951	4	0.024	2	2007	12	8
3	1	112.951	113.623	4	0.672	2	2007	12	8

Source: University of Utah

Figure 120. Chart. Location information in Minnesota's HSIS Roadway File.

Minnesota Step 2: Locate the same point in Minnesota's GIS roadway file and calculate its coordinates

Minnesota's GIS roadway data file also includes route number, county number, and milepost information. The point of interest (Route 1, County 4, Milepost 110.453) can be located and highlighted by opening the attribute table in ArcMap™ software. Figure 121 is a screenshot that illustrates this process. The eighth row (yellow-highlighted) has the beginning milepost of 110.453, the point of interest in this example. Two new columns named Beg_X and Beg_Y are created to store the x and y coordinates of the point (milepost 110.453). The team used the Calculate Geometry tool in ArcMap™ to transform the coordinate information of the point from its original NAD 1983 UTM (Zone 15N) to longitude and latitude in the WGS84 Coordinate System.

RTE_SYST	RTE_NUM	BEGM	ENDM	CNTY_CODE	DIRECTIONA	Beg_X	Beg_Y
03	1	89.364	91.385	04	0300000001-I	-95.247848	48.121582
03	1	91.385	93.712	04	0300000001-I	-95.247817	48.092278
03	1	93.712	94.841	04	0300000001-I	-95.256551	48.059274
03	1	94.841	96.003	04	0300000001-I	-95.267154	48.04476
03	1	96.003	96.466	04	0300000001-I	-95.276589	48.029191
03	1	96.466	96.746	04	0300000001-I	-95.280723	48.022963
03	1	110.124	110.453	04	0300000001-I	-95.193407	47.870257
03	1	110.453	112.857	04	0300000001-I	-95.185892	47.870113
03	1	112.857	113.718	04	0300000001-I	-95.134993	47.869213
03	1	113.718	114.138	04	0300000001-I	-95.116253	47.869173
03	1	114.138	115.74	04	0300000001-I	-95.107424	47.869309
03	1	115.74	116.572	04	0300000001-I	-95.072957	47.868924
03	1	116.572	116.711	04	0300000001-I	-95.053742	47.869084
03	1	116.711	117.026	04	0300000001-I	-95.052118	47.869076
03	1	117.026	117.127	04	0300000001-I	-95.045169	47.869123

Source: University of Utah

Figure 121. Chart. Location information of the roadway segment in GIS File (Minnesota).

Minnesota Step 3: Locate and mark the point in Google Earth™

The team transferred the coordinates calculated from Step 2 into Google Earth™. A marker was placed at the location for future reference. Figure 122 is a screenshot of the point (MN Route 1, County 4, Milepost 110.453) located and marked with a place marker in Google Earth™.

Original image: ©2014 Google®; annotations by University of Utah.

Figure 122. Photo. Locate and mark the point of interest in Google Earth™ using its coordinates (Minnesota).[84]

The above step-by-step process is systematically performed for all beginning points of the road segments on each route of interest in Minnesota. The coordinates of all points are then exported and automatically transferred into Google Earth™ using Keyhole Markup Language (KML) codes. The result of this process is a longitudinal reference system along each route in Google Earth™ with mileposts marked.

Illinois Step 1: Collect route number, county number, and milepost from the HSIS roadway data file

This step is very similar to Step 1 for Minnesota because the HSIS roadway data files from the two States have similar structures and identical information for county number, State route number, and milepost. However, GIS data files from Illinois do not have milepost information. Therefore, an additional piece of information is necessary for this step: the milepost of the route at the county line. The county line is then used as a reference point to calculate mileposts and locate all other points. Figure 123 shows an example of location information found in the Illinois HSIS roadway data file. Two rows in this figure are highlighted. The sixth row (green highlighted) is the first roadway segment on Route 24 in Brown County. The beginning milepost of this segment is the milepost of Route 24 at the county line (begmp = 31.54). Milepost 31.54 then becomes the reference point for all other points on Route 24 in Brown County. The beginning milepost of the tenth row (yellow highlighted) is the point of interest in this example. It is located in County 5 (Brown County), on Route 24, at milepost 33.06.

county	surf_wid	s_rtenbr	no_lanes	lanewid	muni_name	begmp	seg_lng	endmp
1	22	24	2	11	ADAMS	30.4	0.02	30.42
1	22	24	2	11	ADAMS	30.42	0.04	30.46
1	22	24	2	11	ADAMS	30.46	0.04	30.5
1	22	24	2	11	ADAMS	30.5	0.08	30.58
1	22	24	2	11	ADAMS	30.58	0.96	31.54
5	22	24	2	11	BROWN	31.54	0.36	31.9
5	22	24	2	11	BROWN	31.9	0.08	31.98
5	22	24	2	11	BROWN	31.98	0.28	32.26
5	22	24	2	11	BROWN	32.26	0.8	33.06
5	22	24	2	11	BROWN	33.06	0.66	33.72
5	22	24	2	11	BROWN	33.72	0.07	33.79
5	22	24	2	11	BROWN	33.79	0.02	33.81
5	22	24	2	11	BROWN	33.81	0.59	34.4
5	22	24	2	11	BROWN	34.4	0.22	34.62
5	22	24	2	11	BROWN	34.62	0.04	34.66

Source: University of Utah

Figure 123. Chart. Location information in Illinois HSIS Roadway data file.

Illinois Step 2: Locate the same point in Illinois GIS roadway file

Again, unlike Minnesota data, GIS files for the Illinois roadway network do not include milepost information. Illinois GIS files have two variables named BEG_STA and END_STA representing the stations (in units of mi) of beginning and ending points of each GIS roadway segment. (It should be noted that Illinois GIS deviates from standard surveying station measurements, which are usually in increments of 100 ft). Although actual milepost information is not available, the station can be used to compute the milepost of any point along the route. In most cases, BEG_STA=0.00 at the county line, and it increases west to east or south to north throughout the county. In some cases, the station resets back to 0.00 at some major intersecting points with other major routes.

The milepost of any given point on an Illinois route is computed using the equation shown in figure 124.

$$Milepost = Milepost_{ref} + STA$$

Figure 124. Equation. Reference point milepost.

Where:

$Milepost_{ref}$ = the milepost of the reference point
STA = the station information (BEG_STA or END_STA) of the point in the GIS data file

In most cases, the milepost of the reference point is the milepost at the county line (determined in step 1). In some cases, it is the milepost of a major intersection where the beginning station (BEG_STA) resets back to 0.00. This reference point can be identified by looking through the attribute table of the GIS file and locating it using the same procedure as that used to compute mileposts.

While the direction of increasing milepost is the same as the direction of increasing station in the GIS files in most cases, they can be opposite in some rare instances. Because the distances are still the same, mileposts can be computed by reversing the direction of increasing station using the equation shown in figure 125.

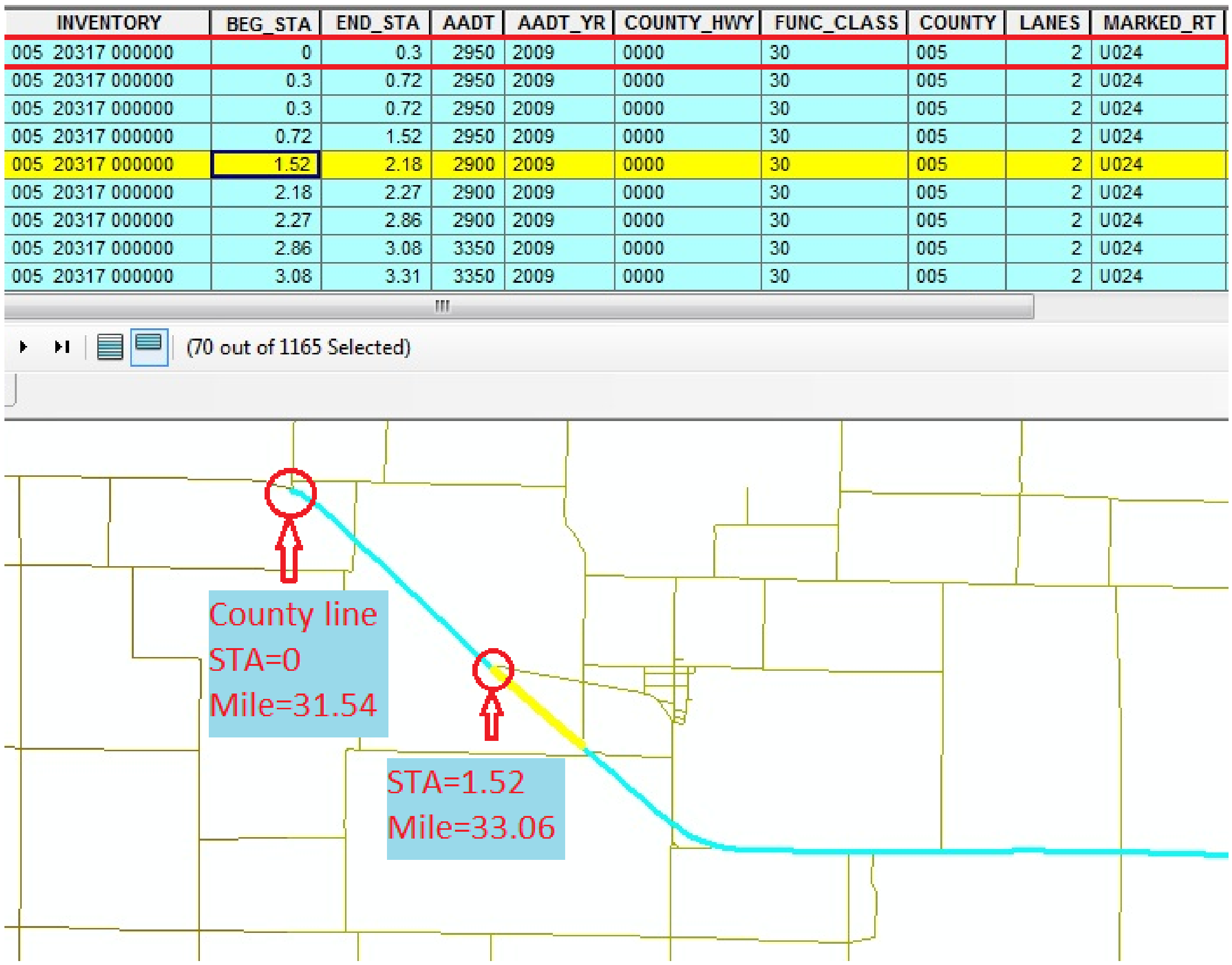
$$Milepost = Milepost_{ref} + (STA_{max} - STA)$$

Figure 125. Equation. Reference point milepost by reversing the direction of increasing station.

Where:

$Milepost_{ref}$ = the milepost of the reference point
STA_{max} = the largest value of END_STA in the GIS file of a given county
STA = the station information (BEG_STA or END_STA) of the point in the GIS data file

Figure 126 shows an example of identifying the point of interest in this example. The first row of the data table indicates the county line with BEG_STA=0.00. The fifth row (yellow highlighted) shows the point of interest with a beginning station of 1.52 at milepost 33.06.



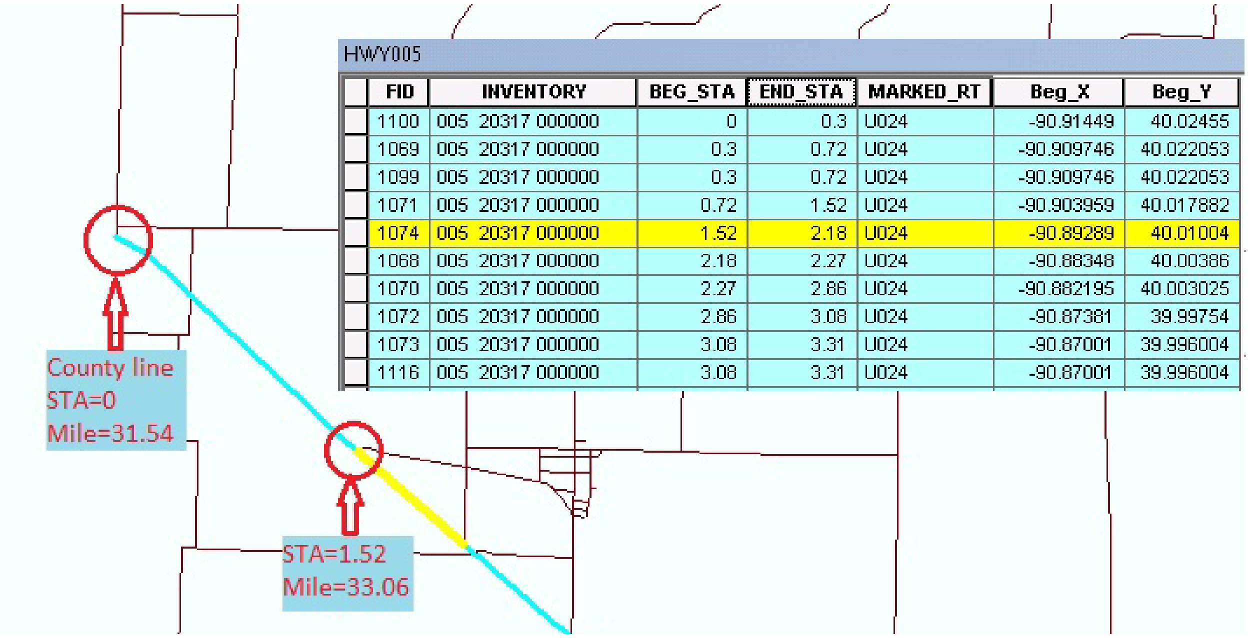
INVENTORY	BEG_STA	END_STA	AADT	AADT_YR	COUNTY_HWY	FUNC_CLASS	COUNTY	LANES	MARKED_RT
005 20317 000000	0	0.3	2950	2009	0000	30	005	2	U024
005 20317 000000	0.3	0.72	2950	2009	0000	30	005	2	U024
005 20317 000000	0.3	0.72	2950	2009	0000	30	005	2	U024
005 20317 000000	0.72	1.52	2950	2009	0000	30	005	2	U024
005 20317 000000	1.52	2.18	2900	2009	0000	30	005	2	U024
005 20317 000000	2.18	2.27	2900	2009	0000	30	005	2	U024
005 20317 000000	2.27	2.86	2900	2009	0000	30	005	2	U024
005 20317 000000	2.86	3.08	3350	2009	0000	30	005	2	U024
005 20317 000000	3.08	3.31	3350	2009	0000	30	005	2	U024

Source: University of Utah

Figure 126. Chart. Location information of the roadway segment in GIS file (Illinois).

Illinois Step 3: Determine the coordinates and locate the point in Google Earth™

This step is similar to Minnesota step 2 and step 3. The research team used the Calculate Geometry tool in ArcGIS™ to coordinate each point in the WGS 1984 Coordinate System. Figure 127 shows the example of obtaining the coordinate information of milepost 33.06 (STA=1.52) along Illinois Route 24. The team then transferred the coordinates into Google Earth™, and the points were marked in the same way. Figure 128 is a Google Earth™ screenshot with the marker of milepost 33.06.

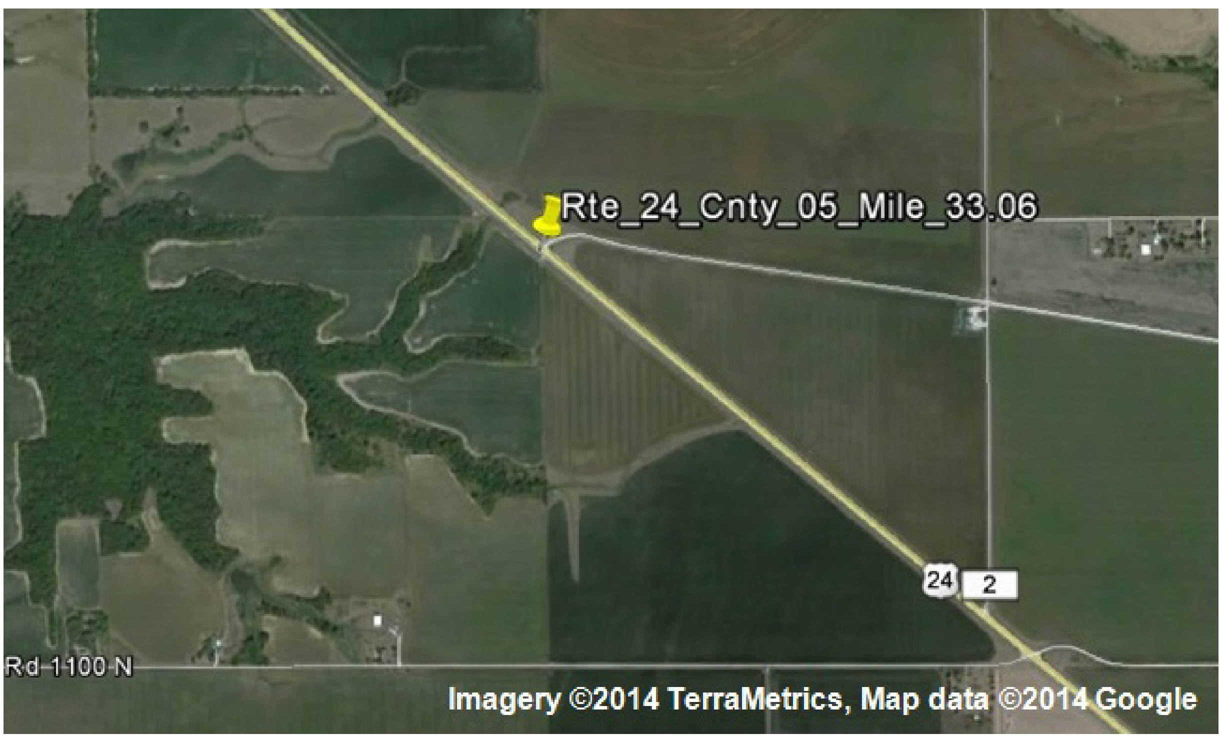


HWY005

	FID	INVENTORY	BEG_STA	END_STA	MARKED_RT	Beg_X	Beg_Y
	1100	005 20317 000000	0	0.3	U024	-90.91449	40.02455
	1069	005 20317 000000	0.3	0.72	U024	-90.909746	40.022053
	1099	005 20317 000000	0.3	0.72	U024	-90.909746	40.022053
	1071	005 20317 000000	0.72	1.52	U024	-90.903959	40.017882
	1074	005 20317 000000	1.52	2.18	U024	-90.89289	40.01004
	1068	005 20317 000000	2.18	2.27	U024	-90.88348	40.00386
	1070	005 20317 000000	2.27	2.86	U024	-90.882195	40.003025
	1072	005 20317 000000	2.86	3.08	U024	-90.87381	39.99754
	1073	005 20317 000000	3.08	3.31	U024	-90.87001	39.996004
	1116	005 20317 000000	3.08	3.31	U024	-90.87001	39.996004

Source: University of Utah

Figure 127. Chart. Identify coordinate information associated with the roadway segment (Illinois).



Original image: ©2014 Google®; annotations by University of Utah.

Figure 128. Photo. Locate and mark the roadway segment in Google Earth™ with its coordinates.[85]

Segment Definition

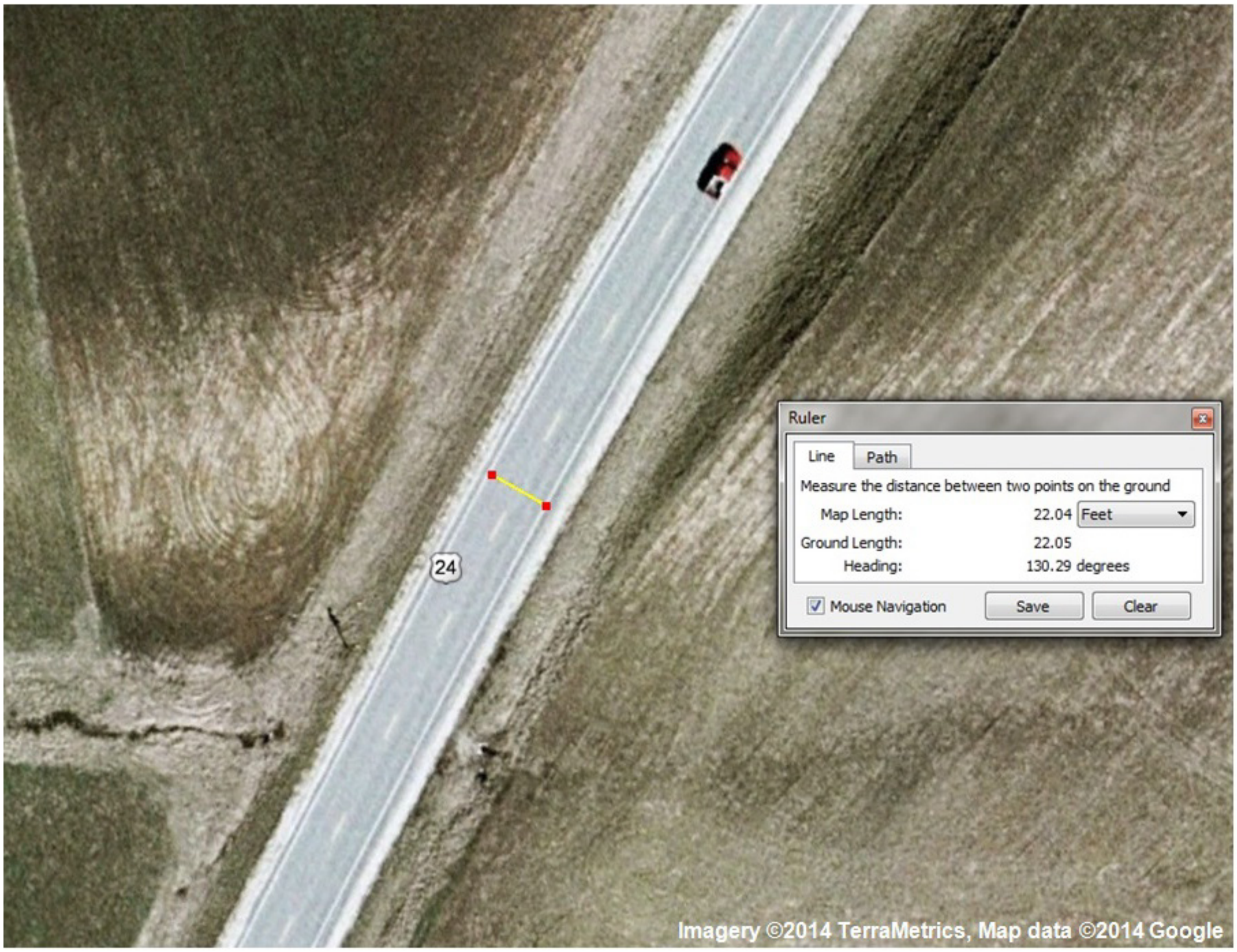
Following the steps outlined above, the research team created a series of mile markers in Google Earth™—creating a longitudinal reference system along each route. The team then explored each route over the 3 years of the analysis period (2007, 2008, 2009) to determine the candidate study segments. In the process, the team also checked whether there were any significant changes to the route (indicating a possible reconstruction project that changed milepost definitions). The team used historical satellite imagery (in Google Earth™) and crash data codes to eliminate sections of roadway with work zones during the analysis period. The team selected and marked candidate study segments based on the following three criteria:

- The segment does not include a signalized or an unsignalized intersection.
- The segment can include driveways.
- The number of lanes, lane width, shoulder width, shoulder type, and traffic volume are the same throughout the segment.

Through this process, the team retained only longer segments (about 0.5 mi and longer). As explained above, the purpose of using the longer segments is to address a limitation of previous studies and capture results regarding the proposed idea that using geometry as a speed management technique will appear effective from a safety perspective only when applied and studied over a significant stretch of road.

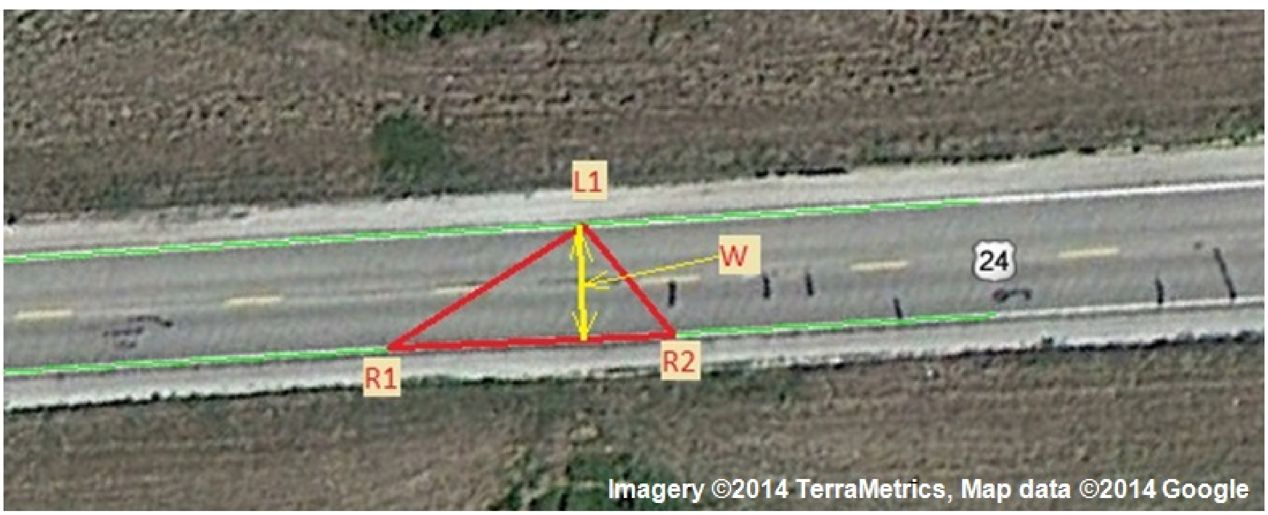
Cross-Section Features

Using the HSIS roadway files, the research team had a basic idea of the lane configuration (i.e., number of through lanes and auxiliary lane present), lane width, shoulder width, and shoulder types. The team then used a combination of Google Earth™ satellite images and a set of Google Earth™ calculation tools created by the research team using Visual Basic for Applications (VBA) in Microsoft™ Excel to check these features at incremental distances along the defined segments. Figure 129, figure 130, and figure 131 are screenshots of Google Earth™ and Google Street View™ illustrating the procedure. The team tested the accuracy of the Google Earth™ measurements of widths using Google Earth™'s distance measurement tools to take measurements of known distances (e.g., football field markings), which were found to be accurate. The determination of lane width was partly automated by developing VBA codes to continuously calculate the height of a moving triangle along each segment. Two points on one edge line and another on the opposite edge of the road form the triangle. Figure 131 illustrates this concept.



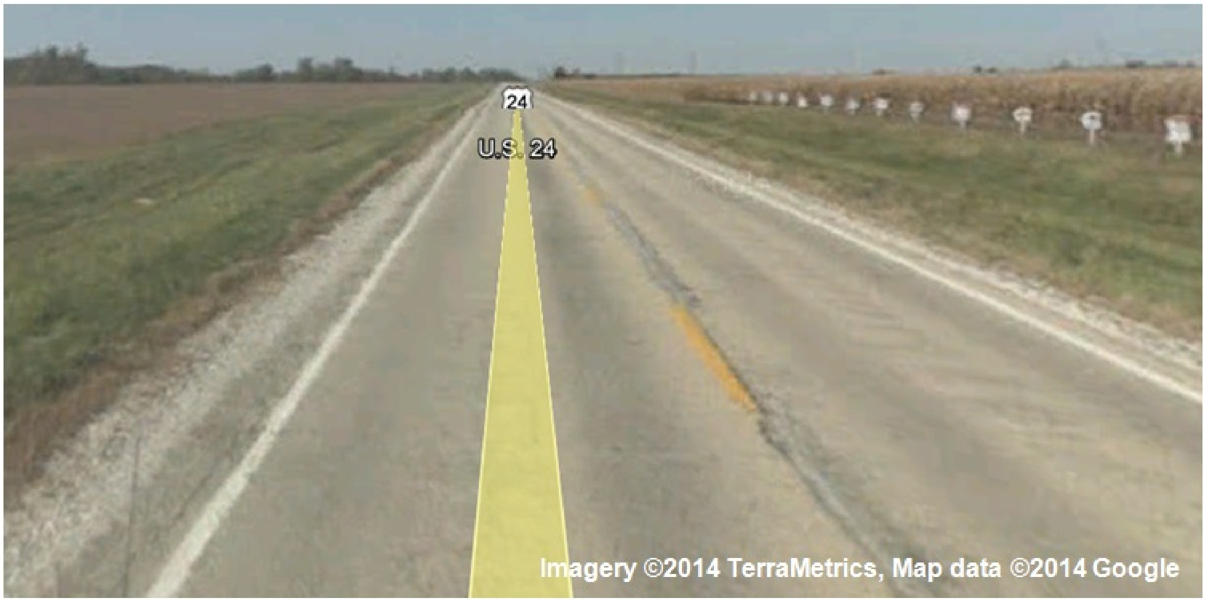
Original image: ©2014 Google®; annotations by University of Utah.

Figure 129. Photo. Measurement of cross-section features in Google Earth™.(86)



Original image: ©2014 Google®; annotations by University of Utah.

Figure 130. Photo. Conceptual illustration of calculating lane.(87)



Original image: ©2014 Google®; annotations by University of Utah.

Figure 131. Photo. Visual checks of cross-section features in Google Street View™.(88)

Other roadside elements may also influence safety and should be controlled for when trying to determine the safety effects of lane-width–shoulder-width combinations. These include barrier presence, barrier length and offset (from the travel lane), type of center line and edge line markings, number of driveways, and the roadside hazard rating. The team adopted the roadside hazard rating developed by Zegeer et al. and currently used in the HSM predictive method for rural, two-lane roads for this study.(89) The team collected these data elements and then coded for the defined segments.

Table 97 provides a summary of the dataset by cross-section configuration with the number of roadway segments by each lane-width and shoulder-width category.

Table 97. Number of segments by lane-width and shoulder-width.

Lane Width (ft)	Shoulder Width (ft)	State		Total
		Illinois	Minnesota	
10	**All**	**55**	**6**	**61**
	0–3	38	0	38
	4–6	15	2	17
	> 6	2	4	6
11	**All**	**267**	**81**	**348**
	0–3	109	9	118
	4–6	142	16	158
	> 6	16	56	72
12	**All**	**219**	**249**	**468**
	0–3	57	28	85
	4–6	126	53	179
	> 6	36	168	204
13	**All**	**0**	**9**	**9**
	0–3	0	6	6
	4–6	0	1	1
	> 6	0	2	2
Total		**541**	**345**	**886**

Table 98 provides a summary of the dataset by the presence of roadside barriers; table 99 shows the presence of driveways.

Table 98. Number of segments with roadside barriers.

Roadside Barriers	State		Total
	Illinois	Minnesota	
No	416	300	716
Yes	125	45	170
Total	**541**	**345**	**886**

Table 99. Number of segments by driveway presence.

Number of Driveways	State		Total
	Illinois	Minnesota	
0	134	95	229
1–10	368	242	610
11–20	30	8	38
> 20	9	0	9
Total	**541**	**345**	**886**

Horizontal Alignment

A limitation of previous studies of lane-width–shoulder-width combinations was the lack of control for horizontal curvature, particularly if the data were from States other than Washington. Horizontal curve data are not available in most roadway inventory databases, but these data are crucial to modeling safety performance along rural and suburban roads. The research team used a combination of Google Earth™ path-marking tools and custom-made Google Earth™ calculation tools developed using VBA in Microsoft™ Excel to collect data for each horizontal curve on the defined road segments. The method was developed and tested using horizontal curves of known geometry and found to be accurate (i.e., measured and actual radius within approximately 2.5 percent). The following steps illustrate this technique, which measures the deflection angle and long chord.

Step 1: Place a path on the centerline. Use the Add path tool in Google Earth™ to trace along the centerline of roadway. This path needs to cover the entire curve and extend to partly cover both tangent lines. This path is visually placed and adjusted so that it matches the centerline of the roadway as closely as possible (see figure 132).

Step 2: Export the overlaid Google Earth™ path and extract the coordinate information. The path is then exported from Google Earth™ into a KML file. The KML file holds coordinate information of points along this path. These coordinates are then extracted using VBA codes specifically developed for this task. A computer algorithm also developed specifically for this task detects and categorizes these points into those on the curve and those on the two tangents based on their coordinates and positions relative to each other.

Step 3: Develop straight-line equations for the tangents and compute the deflection angle. The VBA program then uses the coordinates of the points on the tangents to compute two straight-line equations through linear regression. The coordinates of point of intersection (PI) and the deflection angle (Δ) are then computed from these two straight-line equations.

Original image: ©2014 Google®; annotations by University of Utah.

Figure 132. Photo. Screenshot of estimating deflection angles in Google Earth™.(90)

Step 4: Calculate external distance *E*, radius *R,* and length of curve *L*. With the coordinates of the PI known, a minimization algorithm determines the smallest distance between the PI and the curve. The computer code searches for the smallest distance between the PI and all points along the curve using the equation shown in figure 133.

$$E = Min(distance\ from\ PI\ to\ all\ points\ on\ curve)$$

Figure 133. Equation. Smallest distance between PI and all points along the curve.

The curve radius is then calculated using the equation shown in figure 134.

$$R = \frac{E}{\left(\frac{1}{cos\left(\frac{\Delta}{2}\right)} - 1\right)}$$

Figure 134. Equation. Curve radius.

The length of the curve is then calculated using the equation shown in figure 135.

$$L = \frac{2\pi \times \Delta \times R}{360}$$

Figure 135. Equation. Length of the curve.

This four-step process is applied to every horizontal curve along the defined study segments. The team explored alignment indices to capture changes in horizontal alignment to define longer segments with multiple curves in the database. The advantages of this approach are discussed earlier in this section. Table 100 provides alternative indices explored as ways to quantify the horizontal alignment characteristic of each extended road segment. These are based on a previous study that attempted to use alignment indices to predict operating speeds.[(91)] It is important to note that the objective of this study is not to estimate the effects of the alignment indices on speed and safety, but to control for horizontal alignment characteristics on extended lengths of road while studying the effects of lane-width–shoulder-width combinations.

Table 100. Alignment indices for quantifying horizontal alignment characteristics (based on Fitzpatrick et al., 2000).[91]

Horizontal Alignment Index	Formula
Curvature Change Rate (CCR) (degrees/mi)	$\frac{\sum I_i}{\sum L_i}$ Where: I = deflection angle (degrees) L = length of section (mi)
Degree of Curvature (DC) (degrees/mi)	$\frac{\sum (DC)_i}{\sum L_i}$ Where: DC = degree of curvature (degrees) L = length of section (mi)
Curve Length: Roadway Length (CL:RL)	$\frac{\sum (CL)_i}{\sum L_i}$ Where: CL = curve length (mi) L = length of section (mi)
Average Radius—AVG R (ft)	$\frac{\sum (R)_i}{n}$ Where: R = radius of curve (ft) n = number of curves within section
Average Tangent—AVG T (ft)	$\frac{\sum (TL)_i}{n}$ Where: TL = tangent length (ft) n = number of tangents within section

Other Features

The research team also used maps with area type boundaries combined with the Google Earth™ images and Google Street View™ photography to collect other variables that describe the road segment. These include area type (rural, rural transition zone, rural community, suburban), presence and type of edge line and centerline markings, and presence of shoulder and/or centerline rumble strips.

Table 101 shows a summary of the dataset by the presence of no-passing zone (solid centerline), one-directional passing zone (one-sided solid centerline), and the presence of shoulder rumble strips. Rumble strip information was initially included in the model. However, most of the segments with shoulder rumble strips are in Minnesota so the rumble strip variable is correlated with the State indicator. Therefore, the research team dropped rumble strips from the model, and the State indicator likely captured its effects.

Table 101. Number of segments with no-passing zone, one-sided passing zone, and shoulder rumble strip.

Segment Characteristics		State		Total
		Illinois	**Minnesota**	
No Passing (Solid centerline)	No	382	272	654
	Yes	159	73	232
	Total	**541**	**345**	**886**
One-Sided Passing (One-sided solid centerline)	No	246	164	410
	Yes	295	181	476
	Total	**541**	**345**	**886**
Shoulder Rumble Strip	No	536	180	716
	Yes	5	165	170
	Total	**541**	**345**	**886**

TRAFFIC DATA

The team collected traffic volumes from the HSIS database. Traffic volume was one of the variables used to define a homogenous roadway segment. The team defined traffic volume for each roadway segment for each of the three analysis years (2007, 2008, 2009).

Table 102 provides descriptive statistics for the geometric, traffic control, and traffic variables that describe the 886 rural, two-lane highway segments observed for this study. Ultimately, the approximately 1 percent of segments (a total of 9 segments) with lane widths equal to 13 ft were deleted from the database used for model estimation, resulting in 877 total segments.

Table 102. Descriptive statistics for geometric, traffic control, and traffic variables.

Variable[a]	Number of Observation	Mean	Standard Deviation	Minimum	Maximum
aadt	886	3,436.54	2,353.04	450	19,190
ln_aadt	886	7.93	0.67	6.11	9.86
seglen	886	0.82	0.41	0.11	2.92
illinois	886	0.61	0.49	0	1
Lane10	886	0.07	0.25	0	1
Lane11	886	0.39	0.49	0	1
Lane12	886	0.53	0.50	0	1
Lane13	886	0.01	0.10	0	1
shoulder	886	5.29	2.78	0	12.00
ln10shld	886	0.22	1.02	0	9.00
ln11shld	886	1.84	2.79	0	12.00
barrier	886	0.01	0.02	0	0.36
drvwy_den	886	3.93	5.40	0	63.41
solid_CL	886	0.07	0.17	0	1.13
dash1_CL	886	0.10	0.12	0	0.50
curve	886	0.30	0.46	0	1

[a]The following are the variable definitions:
aadt = Average annual daily traffic (vehicles/day)
ln_aadt = Natural logarithm of aadt
seglen = Segment length (mi)
illinois = Indicator for State in which the road segment is located (=1 if the data is from Illinois; =0 if the data is from Minnesota)
Lane10 = Indicator for 10-ft lane width (=1 if the segment has a 10-ft lane width; =0 otherwise)
Lane11 = Indicator for 11-ft lane width (=1 if the segment has a 11-ft lane width; =0 otherwise)
Lane12 = Indicator for 12-ft lane width (=1 if the segment has a 12-ft lane width; =0 otherwise)
Lane13 = Indicator for 13-ft lane width (=1 if the segment has a 1-ft lane width; =0 otherwise)
shoulder = Total shoulder width, both paved and unpaved shoulders (ft)
ln10shld = Interaction between 10-ft lane and shoulder width (=Lane10*Shoulder)
ln11shld = Interaction between 11-ft lane and shoulder width (=Lane11*Shoulder);
barrier = Length and offset of roadside barrier (=barrier_length/(barrier_offset*total_segment_length)
drwy_den = Driveway density (=number_of_driveways/total_segment_length)
solid_CL = Variable representing the proportion of no-passing zone (= total_length_of_solid_center_line/total_segment_length)
dash1_CL = Variable representing the proportion of one-sided passing zone (=total_length_of_one-sided_solid_center_line/total_segment_length)
curve = Indicator variable representing the horizontal alignment (=1 if the total sum of (radius/deflection angle) falls between 10 and 1,000; =0 otherwise).

CRASH DATA

The team collected crash data from the HSIS crash databases. After defining all study segments and determining their beginning and ending mileposts, the team counted the number of crashes that occurred within each segment. The team used 3 years of crash data (2007, 2008, 2009) for the safety analysis in this study. The team also counted the following crash types on each road segment:

- Total crashes (all types and severities).
- Fatal-plus-injury crashes (all types).
- Single-vehicle crashes (all severities).
- Multiple-vehicle crashes (all severities).
- Key lane width/shoulder with crashes (all severities) that include single-vehicle-run-off-road and multiple-vehicle head-on, sideswipe opposite direction, and sideswipe same direction.
- Single-vehicle crashes (fatal-plus-injury).
- Multiple-vehicle crashes (fatal-plus-injury).
- Key lane width/shoulder with crashes (fatal-plus-injury).
- Single-vehicle crashes (property damage only (PDO)).
- Multiple-vehicle crashes (PDO).
- Key lane width/shoulder with crashes (PDO).

Table 103 provides descriptive statistics for the crash variables.

Within the total and fatal-plus injury crash levels, the team explored crash severity in greater detail because of the expected interaction among lane width, shoulder width, and speed discussed in the background section. The team estimated SDFs for the total and fatal-plus-injury crashes using multinomial logit models (discussed at greater depth in the following section). The databases used to estimate the severity models consisted of the same crashes, road segments, and variables as the frequency model databases, but were restructured so the basic observation unit (i.e., database row) is the crash instead of the road segment.

Table 103. Descriptive statistics for crash variables.

Variable	Number of Observation	Mean	Standard Deviation	Minimum	Maximum
tot_all	886	2.71	3.55	0	32
sv_all	886	2.27	3.21	0	31
mv_all	886	0.44	0.94	0	10
key_kabco	886	2.94	4.01	0	35
sv_fi	886	0.31	0.66	0	6
mv_fi	886	0.17	0.47	0	4
key_kabc	886	0.57	1.14	0	10
sv_o	886	1.95	2.96	0	28
mv_o	886	0.27	0.71	0	7
key_o	886	2.38	3.49	0	30

The following are the variable definitions:
tot_all = Total crashes (all types and severities)
sv_all = Single-vehicle crashes (all severities)
mv_all = Multiple-vehicle crashes (all severities)
key_kabco = Key lane width/shoulder with crashes (all severities) that include single-vehicle-run-off-road and multiple vehicle head-on, sideswipe opposite direction, and sideswipe same direction
sv_fi = Single-vehicle crashes (fatal-plus-injury)
mv_fi = Multiple-vehicle crashes (fatal-plus-injury)
key_kabc = Key lane width/shoulder with crashes (fatal-plus-injury)
sv_o = Single-vehicle crashes (property damage only (PDO))
mv_o = Multiple-vehicle crashes (PDO)
key_o = Key lane width/shoulder with crashes (PDO)

SAFETY DATA ANALYSIS METHODOLOGIES

Analysis of Expected Crash Frequency

The team used negative binomial regression modeling to explore the effects of lane-width–shoulder-width combinations on expected crash frequency. The team estimated the full regression models that consider all the variables collected, in addition to lane width and shoulder width, to reduce the possibility of omitted variable bias. In the negative binomial model, the expected number of crashes of type i on segment j is typically expressed using the equation shown in figure 136.

$$\mu_{ij} = E\left(Y_{ij}\right) = e^{\left(X_j\beta + ln\left(L_j\right)\right)}$$

Figure 136. Equation. Expected number of crashes of type i on segment j.

Where:

$\mu_{ij} = E(Y_{ij})$ = the expected number of crashes of type *i* on segment *j*
X_j = a set of traffic and geometric variables characterizing segment *j*
β = regression coefficients estimated with maximum likelihood that quantify the relationship between $E(Y_{ij})$ and variables in X

L_j = length of segment j
$Ln(L_j)$ = the natural logarithm of segment length

The mean-variance relationship of the negative binomial regression model is expressed using the equation shown in figure 137.

$$VAR(Y_{ij}) = E(Y_{ij}) + \alpha[E(Y_{ij})]^2$$

Figure 137. Equation. Variance of crashes of type i on segment j.

Where:

$VAR(Y_{ij})$ = variance of crashes of type i on segment j
$E(Y_{ij})$ = the expected number of crashes of type i on segment j
α = overdispersion parameter

The research team explored both constant overdispersion parameters as well as overdispersion parameters that apply to a unit length of road when estimating the lane-width and shoulder-width models. Ultimately, they selected models with a constant overdispersion parameter.

The research team explored variations of the negative binomial model to address crash-influencing factors that may be common to groups of road segments, but that are not captured by the models. Failure to address these factors during model estimation may result in biased parameter estimates and underestimated standard errors. Hauer classified these factors as "unrecognized, or not understood, or unmeasured."[(92)] Examples include terrain type, weather, and crash-reporting thresholds of the agency overseeing a particular area. The latter can be particularly important in estimating safety performance functions for less severe (e.g., PDO) crashes.

Negative binomial models with identification variables for counties, districts, regions, or States treated as fixed or random effects are modeling alternatives to address these "shared unobservables." The fixed effects estimator has less restrictive assumptions, but it is not always useful if the independent variables of interest do not vary within the fixed effects (e.g., if a particular county had the same lane width throughout, then the fixed effects for that county and lane width could not both be included in the model). The random effects estimator addresses this challenge, but is only appropriate if the random effects are not correlated with the independent variables in the model. This assumption can be investigated with a Hausman test.[(93)] The models reported in this report ultimately implemented a fixed effects approach with indicator variables representing State (i.e., 1 = Illinois, 0 = Minnesota).

The research team tested the effects of lane-width–shoulder-width combinations through a number of variable specifications. It has been common to include lane width and shoulder width in a linear specification. However, the team also explored the inverse of lane width and shoulder width in an attempt to capture an increased effect of lane width and shoulder width on safety when widths are narrow. As the widths become wider and wider, the safety effects diminish. The team also tested indicator variables for lane-width–shoulder-width combinations. Indicator variables capture possible nonlinear or noncontinuous relationships. In the end, the research team settled on specifications of indicator variables for lane widths, shoulder width as a continuous variable in a linear specification, and interactions between the lane-width indicator variables and shoulder width to capture the varying effects of lane-width–shoulder-width combinations.

Analysis of Crash Severity

Accurate predictions of crash severity are important when looking at combinations of lane widths and shoulder widths. Severity distributions may change significantly with cross-section allocation through a resulting increase or decrease in operating speeds. Severity distributions are likely to vary differently with traffic volumes and design decisions depending on crash type (e.g., single vehicle or multiple vehicle). The research team used logit models to estimate an SDF to address these issues. The logit models produce the probabilities (or proportions) of crash severity outcomes as a function of traffic volume, geometry, and other road characteristics. The multinomial logit, nested logit, and ordered outcome models are possible model alternatives.[94,95,96] The databases used to estimate the severity models consist of the same crashes road segments and variables as the frequency model databases, but restructured so that the basic observation unit (i.e., database row) is the crash instead of the road segment. The frequency models and SDFs can be combined to estimate the number of accidents of different severity levels.[97]

For this research, the research team used the multinomial logit model to estimate SDFs within the total and fatal-plus-injury crash levels. In the multinomial logit model, the probability that accident *n* will have severity *i* is expressed using the equation shown in figure 138.

$$\mathrm{p}_n(\mathrm{i})=\exp(\beta_i \mathrm{X}_n)/ \sum_I \exp(\beta_I X_n)$$

Figure 138. Equation. Probability of accident *n* with severity *i*.

Where X_n is a set of variables that will determine the crash severity, and β_i is a vector of parameters to be estimated. The same factors specified in the frequency models were also specified in the severity models, including variables for different lane-width–shoulder-width combinations at the crash location.

MODEL ESTIMATION RESULTS AND DISCUSSION

All models were estimated using STATA® 12.1, created by StataCorp, located in College Station, TX. Table 104 and table 105 provide negative binomial regression model estimation results for total crashes (all types and severities) fatal-plus-injury crashes (all types), respectively. Appendix C provides model estimation results for other crash types (as outlined in the Crash Data section).

Table 104. Negative binomial regression model estimation results for total crashes (all types and severities).

Variable	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
ln_aadt	0.652	0.053	12.25	< 0.001	0.547	0.756
illinois	0.994	0.092	10.78	< 0.001	0.813	1.175
Lane10	0.985	0.201	4.89	< 0.001	0.590	1.380
Lane11	0.489	0.146	3.34	0.001	0.202	0.775
shoulder	-0.042	0.021	-1.97	0.048	-0.083	0.000
ln10shld	-0.119	0.053	-2.25	0.024	-0.222	-0.016
ln11shld	-0.052	0.028	-1.88	0.060	-0.106	0.002
barrier	3.925	1.593	2.46	0.014	0.803	7.047
drvwy_den	0.014	0.006	2.32	0.020	0.002	0.025
solid_CL	0.517	0.188	2.75	0.006	0.149	0.886
dash1_CL	0.862	0.293	2.94	0.003	0.288	1.437
curve	0.224	0.067	3.35	0.001	0.093	0.355
_cons	-5.041	0.424	-11.89	< 0.001	-5.872	-4.210
ln(Length)	1	(exposure)		N/A	N/A	N/A
/lnalpha	-1.129	0.117	N/A	N/A	-1.358	-0.900
alpha	0.323	0.038	N/A	N/A	0.257	0.406

Log Likelihood = -1608.0603
Number of Observations = 877
LR chi2(12) = 443.79
Prob > chi2 = 0
Pseudo R2 = 0.1213
Likelihood-ratio test of alpha=0: chibar2(01) = 239.90 Prob>=chibar2 = 0.000
N/A = Not Available

Table 105. Negative binomial regression model estimation results for fatal-plus-injury crashes (All types).

Variable	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
ln_aadt	0.736	0.092	7.980	< 0.001	0.555	0.917
illinois	0.426	0.151	2.830	0.005	0.131	0.721
Lane10	0.315	0.384	0.820	0.412	-0.437	1.066
Lane11	0.424	0.259	1.640	0.101	-0.083	0.932
shoulder	-0.016	0.035	-0.450	0.650	-0.085	0.053
ln10shld	-0.047	0.093	-0.500	0.615	-0.228	0.135
ln11shld	-0.063	0.047	-1.330	0.183	-0.156	0.030
barrier	2.370	2.552	0.930	0.353	-2.631	7.372
drvwy_den	0.002	0.010	0.170	0.866	-0.019	0.022
solid_CL	0.453	0.311	1.460	0.145	-0.156	1.062
dash1_CL	0.404	0.499	0.810	0.419	-0.575	1.383
curve	0.113	0.113	1.000	0.316	-0.108	0.335
_cons	-6.856	0.738	-9.290	< 0.001	-8.303	-5.409
ln(Length)	1	(exposure)	N/A	N/A	N/A	N/A
/lnalpha	-1.49881	0.464812	N/A	N/A	-2.40983	-0.5878
alpha	0.223396	0.103837	N/A	N/A	0.089831	0.55555

Log Likelihood = -744.43799
Number of Observations = 877
LR chi2(12) = 94.08
Prob > chi2 = 0.000
Pseudo R2 = 0.059
Likelihood-ratio test of alpha=0: chibar2(01) = 6.69 Prob>=chibar2 = 0.005
N/A = Not Available

For the total crash model, all model parameters except one were statistically significant at a 95-percent confidence level (i.e., with a probability of a Type I error less than 5 percent). The *p*-value for the interaction of the 11-ft lane width indicator and shoulder width indicated a 6-percent chance of a Type I error. Parameter signs were all generally in the direction expected. Parameters for lane width indicators showed that, with shoulder width ignored, the expected number of total (i.e., all types and severities) crashes increases as lane width decreases. The main effect of shoulder width was a decrease in the expected number of crashes as shoulder width increased. In addition, the interaction of the lane width indicator and shoulder width showed that shoulder width has the greatest effect on safety when the lane width equals 10-ft. Shoulder width also has a greater effect on safety when the lane width is 11-ft than when the lane width is 12-ft. Figure 139 shows these findings are demonstrated in the form of CMFs.

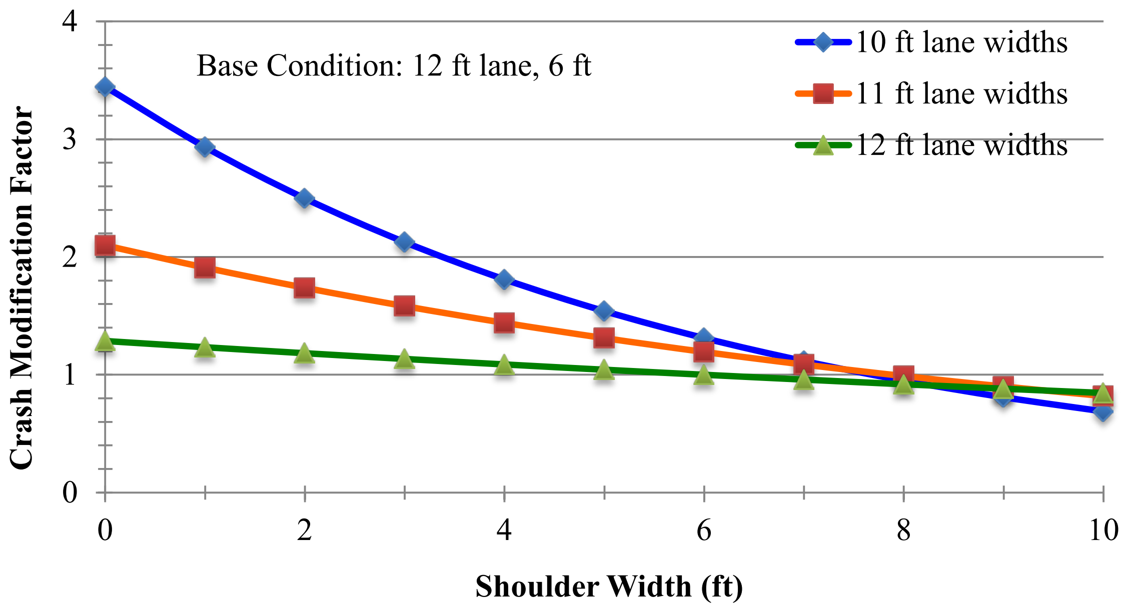


Figure 139. Graph. CMFs for lane-width–shoulder-width combinations developed directly from regression model parameters, total crashes (all types and severities) on rural, two-lane roads (developed from model estimation results in table 104).

For the fatal-plus-injury crashes, only the 11-ft lane width indicator, the 11-ft lane width, and shoulder width interaction were statistically significant at a level higher than 80 percent among the cross-section variables. Parameter signs were all generally in the direction expected, with the same signs as the total crash model parameters. Parameters for lane width indicators showed that, with shoulder width ignored, the expected number of fatal-plus-injury crashes (all types) increases as lane width decreases, but it is difficult to distinguish the performance of an 11-ft lane width from a 12-ft lane width. The main effect of shoulder width was a decrease in the expected number of fatal-plus-injury crashes as shoulder width increased, but the probability of a Type I error was quite high (near 65 percent). The interaction of the lane width indicator and shoulder width showed that shoulder width has additional, positive effects on safety when the lane width is less than 12 ft. Figure 140 shows that these findings are demonstrated in the form of CMFs.

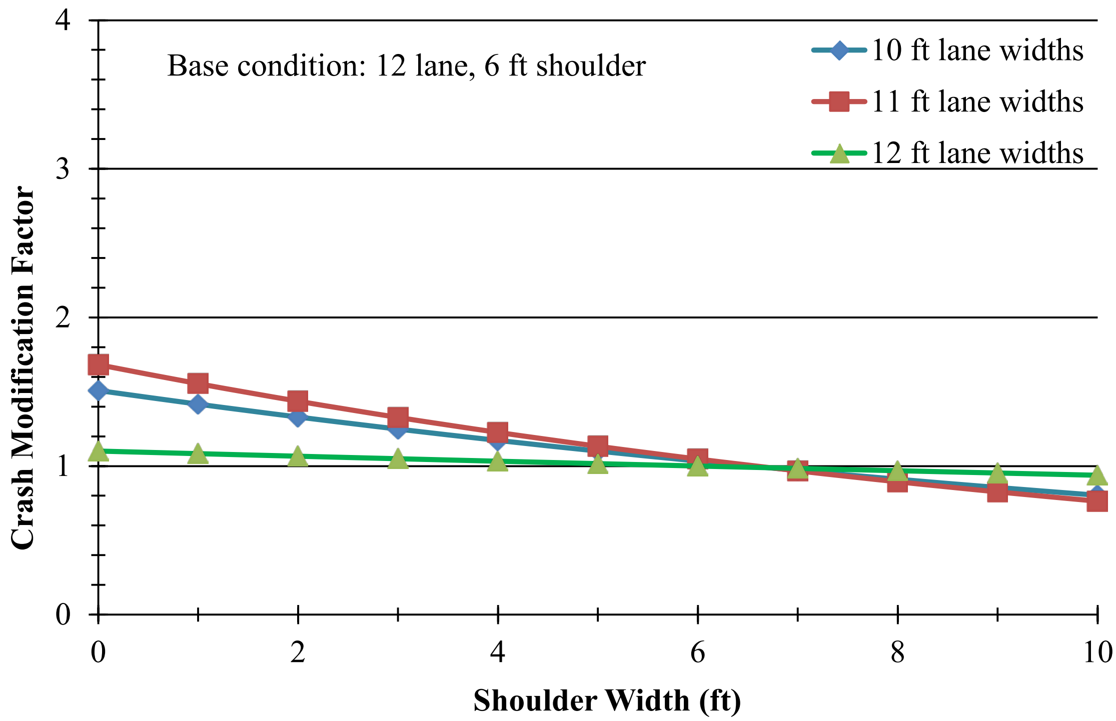


Figure 140. Graph. CMFs for lane-width–shoulder-width combinations developed directly from regression model parameters, fatal-plus-injury crashes (all types) on rural, two-lane roads (developed from model estimation results in table 105).

The results discussed above show more complex (but intuitive) interactions between expected crash frequency, lane width, and shoulder width than what is currently reflected in the *Highway Safety Manual* CMFs. For any given pavement width, there are combinations of lane width and shoulder width that result in the lowest expected crash frequency (for all crash types and severities as well as for fatal-plus-injury crashes). For narrower total paved widths, the optimal lane width appears to be 12 ft. This general conclusion is consistent with conclusions from Gross et al.[48] 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 than expected crash frequencies.

One possible explanation continues to be likely differences in operating speeds for different lane-width–shoulder width combinations. Initially, these safety results were to be linked with observational speed studies conducted along selected Illinois and Minnesota segments to confirm this hypothesis. However, FHWA officials decided to focus on a more general safety study of lane-width–shoulder-width combinations and spend the remaining resources on more extensive field data collection for other treatments studied as part of this research effort (OSBs and high-friction surfaces).

Appendix C provides the multinomial logit model estimation results for total crashes (all types and severities) with property damage only set as the base outcome, and fatal-plus-injury crashes (all types) with possible injury crashes set as the base outcome. This was done in an effort to estimate SDFs in addition to estimating different crash frequency models for various severity levels. The databases used to estimate the severity models consisted of the same crashes and road segments as the frequency model databases, but restructured so the basic observation unit (i.e., database row) is the crash instead of the road segment. The newest *Highway Safety Manual* proposes SDFs in methodologies on freeways and interchanges.

The SDFs do not show statistically significant and consistent results in terms of the geometric effects on crash severity defined in this way. This is not necessarily surprising, given that the interpretation of these models is "the probability of a severity outcome, given that a crash has occurred." The geometric features are not expected to directly influence the severity outcome given that the crash has occurred; geometric features are likely to influence the likelihood of the crash occurring. Vehicle- and occupant-related characteristics, as well as the characteristics of a given collision, are much more likely to influence this conditional severity outcome. The finding indicates the need for additional exploration of SDFs and their sensitivity to road design and traffic control features.

7. SUMMARY, CONCLUSIONS, AND CONSIDERATIONS

This section summarizes project findings, conclusions, and general considerations for future research.

HIGH FRICTION SURFACE TREATMENT EVALUATION FINDINGS

The research team used observational before–after studies to evaluate HFST at four treatment sites and three control sites in West Virginia. The HFST was applied in June 2012 with the first after period data collection in August 2012 and the second after period data collection in June 2013. The research team performed operational, driver behavior, and friction evaluations. The following is a summary of the evaluation findings:

- Operating Speed Evaluation Results
  - **Mean Operating Speed**—The research team compared the mean operating speed for passenger cars and trucks before and at two different time periods after installing the HFST. The mean speeds were compared at a location on the approach tangent, curve PC location, and the midpoint of the horizontal curve. The analysis included only daytime and unopposed free-flow vehicle mean speeds. The results of the mean speed analysis indicate that the HFST did not significantly affect vehicle mean operating speeds in a consistent manner across all four treatment sites. Few of the independent samples *t*-tests showed that the mean passenger car and heavy truck operating speeds differed between the before and first or second after periods at the treatment sites. A multifactor ANOVA found that the data collection time period–treatment site interaction was not statistically significant ($F_{2,\,4160} = 0.49$, $p > 0.05$) at the approach speed location, when including observations of free-flow vehicles only and controlling for the presence of vehicles in the opposing lane, time of day, and vehicle type. Similar results were found at the PC ($F_{2,4160} = 1.51$, $p > 0.05$) and midcurve locations ($F_{2,4160} = 0.56$, $p > 0.05$).
  - **Change in Mean Operating Speed**—The change in speed from the entry of the curve (PC) to the midpoint of the horizontal curve (referred to as delta speed hereafter) was also considered in the analysis. There is no consistent trend in the delta speed for passenger cars from the before to the second after period. There was a significant increase at two treatment sites, but there was also a very significant decrease at another treatment site. A multifactor ANOVA found that the treatment site–time period interaction was not statistically significant ($F_{2,4160} = 0.51$, $p > 0.05$) when including observations of free-flow vehicles only and controlling for the presence of vehicles in the opposing lane, time of day, and vehicle type. This suggests that the HFST did not affect the change in vehicle speeds between the PC and midcurve locations in the current study.
  - **Vehicles Exceeding PSL**—The research team calculated and compared the percentage of vehicles exceeding the PSL and the advisory speed at the treatment and control sites. There were no visible trends from the before period to after

treatment application in the number of passenger cars and heavy trucks observed to exceed the PSL. Alternatively, there appeared to be a slight reduction in the number of trucks exceeding the advisory speed, but there does not appear to be a strong effect on the number of passenger vehicles exceeding the advisory speed.

  - **Speed Variance Analysis**—A two-sided *F*-test was used to compare the variance of vehicle operating speed in the before and after periods for passenger cars and heavy trucks. The effects of the HFST on the speed deviation was considered for the PC and curve midpoint locations because the surface was applied starting at the PC, or just before it, and it was assumed that the surface would have no effect on the speed deviation at the approach data-collection location. There was no consistent trend in the effects of the HFST on the speed deviation of passenger cars. In the first after period, the speed deviation increased at the PC for two sites, but also decreased at two other sites. In the second after period, the speed deviation decreased at the PC for two sites. In the first after period, the speed deviation increased at the curve midpoint at one site, but the decreased at another site. In the second after period, the speed deviation increased at the curve midpoint for two sites, but also decreased at two sites. For trucks, the speed deviation decreased at the PC at one site in both after periods, and also decreased at one site in both after periods. There was no change in speed deviation at the curve midpoint for trucks at any site.

- Encroachment Evaluation Results

  - The purpose of the encroachment evaluation was to determine whether the HFST changed the proportion of vehicles that crossed either the edge line or centerline. There was no significant change in the braking and encroachment data that lasted throughout the treatment period except for an increase in the proportion of vehicles exiting the curve that encroached onto the shoulder at the U.S. 33 treatment site and also a decrease in the proportion of vehicles entering the curve that encroached onto the shoulder at the WV Route 32 treatment site. These results show the HFST does not have a consistent effect on the braking and encroachment behavior of vehicles.

- Friction Evaluation Results

  - The research team used the DF tester and CT meter to evaluate skid resistance of the HFST and comparison site locations. The friction level increased significantly from the before to the after period at the treatment sites, ranging from an increase of 0.21 to 0.30 in the SN65 value. At all four treatment sites, the second after period produced friction levels that were statistically higher than the before period, indicating the higher friction levels produced by the treatment were maintained for at least 1 year. For the margin of safety analysis, the research team estimated the available side (lateral) friction. While the available side friction is inconsistent from before to after at some comparison locations, the change in available side friction at 40 mph is quite substantial at the treatment sites.

- In addition to the friction curves, speed data collected were also used to compute the difference in friction supply and the friction demanded by vehicles traversing horizontal curves at the treatment and control locations. The mean friction demand at all sites, for all time periods, is lower than the mean side friction supply. The 85th percentile friction demand at all treatment sites, for all time periods, except the before time period of the U.S. 33 treatment site, is lower than the mean side friction supply. The 85th percentile friction demand for all time periods of the U.S. 33 comparison site is greater than the side friction supply. The 95th percentile friction demand at all treatment sites, for all time periods, except the before time period of the U.S. 33 treatment site, is lower than the mean side friction supply. The 95th percentile friction demand for all time periods of the U.S. 33 comparison site is greater than the side friction supply. The only additional site where the friction demand at the 95th percentile exceeded the friction supply was at the U.S. Route 219 treatment MM 5.81 site in the before period.

HFST Conclusions

In summary, the operational and driver behavior analyses generally found no consistent differences at the treatment sites between the before and after time periods based on the data collected in the current study. The friction analysis, however, clearly demonstrated that the friction supply increased considerably at the four horizontal curve treatment locations in West Virginia. The friction levels generally remained high for a period of 1 year after the treatment was applied. A safety analysis, currently being completed under a separate FHWA contract, will reveal further information about the safety effects of the HFST.

HFST Considerations for Future Research

The current study collected speed, driver behavior, and friction data at several HFST countermeasure and control sites in West Virginia. An observational before–after safety evaluation is being completed under a separate FHWA project, "Evaluations of Low-Cost Safety Improvements Pooled Fund Study (ELCSI–PFS)." Phase VI of this effort, Safety Performance Evaluation, includes the HFST safety evaluation. Future research is recommended to determine the long-term durability of the HFST by continually assessing the friction supply at the pavement–tire interface. The friction supply should be considered in future safety performance evaluations of this treatment, by considering the time-varying nature of the HFST countermeasure.

OPTICAL SPEED BAR EVALUATION FINDINGS

OSBs were implemented and evaluated at four sites in Arizona, eight sites in Alabama, and seven sites in Massachusetts. Two different designs were tested as part of this research, and a robust data collection and analysis was undertaken to individually track each vehicle thus enabling the determination of speed changes for individual vehicles. After applying the OSBs, the research team collected the first after period data in Arizona in November 2012 and the second after period data in April 2013. In Massachusetts, the research team collected after period data in December 2012. There was no second after period in Massachusetts. In Alabama, the research team collected the first after period data in October 2013 and the second after period data in December 2013. The findings of the evaluation study are as follows:

- **Mean Operating Speeds**—The mean operating speeds at each sensor location for all sites and all data-collection periods was calculated. An initial comparison was made between corresponding speed parameters at each site for each data-collection period by calculating the numerical differences in these speed parameters. The OSBs did not have an effect on the mean operating speeds at any of sites. There were no changes in speeds that were consistent across all of the sites.

- **Speed Difference Analysis**—This measure computed the change in speed (delta speed) from the approach to PC and the change in speed from the control point to the midpoint of the horizontal curve.

    - **Arizona Sites**—Overall, delta speed did not change in the first after period. Delta speeds were significantly different at all sites in the second after period; however, there was no clear trend in the change in delta speed. The change in delta speed varied both in direction and magnitude across the different Arizona sites, with no strong trend present.

    - **Massachusetts Sites**—Unlike the Arizona sites, there was a weak trend in the change of delta speed at the Massachusetts sites. The OSBs may have contributed to a small decrease on both the delta speeds between the approach and PC and between the control point and curve midpoint.

    - **Alabama Sites**—There were many changes in delta speed, but there was no clear trend in the change. Delta speed increased at several sites but also decreased at other sites. Many of the speed changes were a result of control point speeds changing from the before to after periods. These changes at the control point make it difficult to determine whether the OSBs had an effect on delta speed. The change in delta speed varied both in direction and magnitude across the different Alabama sites, with no strong trend present.

- **Analysis of Variance for Speed Differences**—The research team used an ANOVA to compare speed differences in the before and after periods. There was much variability in the speed changes at the different sites in Massachusetts, with the speed differences increasing at some sites, while decreasing at other sites.

- **Vehicles Exceeding the PSL**—The research team calculated the percentage of vehicles exceeding the PSL and the advisory speed at the treatment and control sites and compared between data-collection periods. There was no consistent trend in the change of the proportion of vehicles exceeding the speed limit, meaning the OSBs had no effect.

- **Speed Variance Analysis**—A two-sided *F*-test was used to compare the variances of vehicle operating speeds in the before and after periods for passenger cars and heavy trucks. There was no consistent trend in the change of the SD of speeds at most sites, meaning the OSBs had no effect.

- **Speed Threshold Analysis**—The research team calculated the change in percentage of vehicles traveling a certain threshold over the PSL and compared between data-collection periods. The number of drivers traveling 10 and 15 mph or more over the PSL were compared before and after installation of the treatment. The data did not indicate a consistent reduction in high-end speeding for vehicles traveling over the PSL.

OSB Conclusions

The results of this evaluation yielded inconsistent speed reductions at all the test sites. Based on the results, the following conclusions can be drawn:

- The experimental MCPW design used at seven sites in Massachusetts and four sites in Arizona did not show a consistent trend in speed reductions. Previously, MCPW used a similar design at one site and reported mean speed reductions of 2 mph during daytime and almost 4 mph during nighttime. However, the MCPW study did not track individual vehicles, and only one data collection point was used.

- The design used at eight sites in Alabama did not show reductions in speed. Previous studies that used this design reported minor reductions (1 to 2 mph). However, it is not known whether previous studies employed the same rigorous data collection and analysis as was undertaken by this study (i.e., tracking free-flow vehicles).

Based on the results, it can be concluded that the OSB designs used in this research were unsuccessful in reducing vehicle speeds. The effectiveness of OSBs depends on the bar spacing and marking type, and for this reason, the results of this study cannot be generalized to confirm the ineffectiveness of OSBs.

OSB Considerations for Future Research

From the results of the field trials and previous OSB evaluations, it can be concluded that OSB treatments have little or no influence driver speeds. However, if further studies are undertaken, the following modifications should be considered:

- Marking Type

  - Consider evaluating non-MUTCD (2009 MUTCD, Part 3B.22, recommended markings to be 12 inches in width by 18 inches in length) experimental marking types. For instance, MCPW bar design contained two transverse markings spaced 8 inches apart, and each marking had a 24-inch length by 8-inch width to establish an overall speed bar dimension of 2 ft by 2 ft. Similarly, Iowa State University modified the MUTCD design to include a third bar (in the middle) for more visual effect. The middle bar provides additional visual contrast for the driver and also encourages drivers to place their vehicle between the bars, which is expected to cause drivers to slow as they concentrate on the driving task.

- Evaluate the influence of the OSBs on drivers of different vehicle classes (e.g., trucks).

- Because the beginning of the curve is the most crucial location for influencing driver speeds, consider extending the treatment length such that the OSBs end at the midpoint of the curve.

- Conduct a review of the crash history every 2 to 5 years after installing the markings. This would provide sufficient time to assess the true effects of the OSB on crash performance. In such cases the OSBs must be refreshed to maintain visibility (thermoplastics is preferred over paint).

LANE-WIDTH–SHOULDER-WIDTH COMBINATION EVALUATION FINDINGS

The study also estimated the safety effects of lane-width–shoulder-width combinations on rural two-lane, two-way road segments. Information was collected on 886 segments (minimum length of 0.5 mi) in Illinois (541) and Minnesota (345) with varying lane-width–shoulder-width combinations. Parameters for lane-width indicators showed that, with shoulder width ignored, the expected number of total (i.e., all types and severities) crashes increases as lane width decreases, but it is difficult to distinguish the performance of an 11-ft-lane width from a 12-ft lane width. The main effect of shoulder width was a decrease in the expected number of crashes as shoulder width increased. In addition, the interaction of the lane width indicator and shoulder width showed that shoulder width has the greatest effect on safety when the lane width equals 10 ft. Shoulder width also has a greater effect on safety when the lane width is 11 ft than when the lane width is 12 ft.

Lane-Width–Shoulder-Width Combination Conclusions

The results of the lane-width–shoulder-width safety evaluations show more complex (but intuitive) interactions between expected crash frequency, lane width, and shoulder width than what is currently reflected in the *Highway Safety Manual* CMFs for rural, two-lane roads. For any given pavement width, there are combinations of lane width and shoulder width that result in the lowest expected crash frequency (for all crash types and severities as well as for fatal-plus-injury crashes). For narrower total paved widths, the optimal lane width appears to be 12 ft. This general conclusion is consistent with conclusions from Gross et al.[48] 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.

Lane-Width–Shoulder-Width Combination Considerations for Future Research

FHWA's *A Guide to Developing Quality Crash Modification Factors* indicates that similar conclusions from different cross-sectional studies may be one way to increase confidence in CMFs derived from regression models.[81] Two separate studies, this effort together with Gross et al., have come to similar conclusions regarding lane-width–shoulder-width interactions on rural, two-lane roads using data from different States and different statistical analysis methods.[48] The findings for a safety optimal lane width for a given total paved width imply an underlying interaction between roadway cross-section dimensions, speed, and safety. No study has been able to quantify this interaction to date, although the need to consider users' speed adaptation in safety evaluations is recognized as a high-priority research topic.[98] A before–after safety evaluation, with participating agencies willing to reallocate pavement width to different lane-width–shoulder-width combinations, may offer additional insight. Ideally, this study would also track driver adaptation over time to the lane width and shoulder width changes, specifically with respect to driver speed. This additional piece of information would enhance the interpretation of the safety findings. Increasingly robust cross-sectional studies (e.g., larger samples, additional States) are recommended in the absence of such a before–after study. Supplementing the cross-sectional safety data with a speed data-collection effort across a variety of lane-width–shoulder-width combinations would again strengthen the interpretation of the safety findings.

APPENDIX A. CRASH MODIFICATION FACTORS FOR GEOMETRIC DESIGN FEATURES

Information for table 106 was gathered from the AASHTO *Highway Safety Manual* (2010).(37)

Table 106. CMFs for Geometric Design Features

CMF	Area Type	Facility Type	Notes	Expected CMF (standard error)
Modify Lane Width	Rural	Two lane	None	See figure 13-1 in HSM
Modify Lane Width	Rural	Undivided multilane	None	See figure 13-2 in HSM
Modify Lane Width	Rural	Divided multilane	None	See figure 13-3 from HSM
Modify Lane Width	Rural	Frontage road	None	See figure 13-4 in HSM
Add or Widen Paved Shoulder	Rural	Two lane	None	See figure 13-5 from HSM
Add or Widen Paved Shoulder	Rural	Multilane	8- to 6-ft shoulder	1.04 (N/A)
			8- to 4-ft shoulder	1.09 (N/A)
			8- to 2-ft shoulder	1.13 (N/A)
			8- to 0-ft shoulder	1.18 (N/A)
Add or Widen Paved Shoulder	Rural	Frontage Road	None	See figure 13-6 in HSM
Provide a Raised Median	Rural	Multilane	Injury crashes	0.88 (0.03)
			Non-injury crashes	0.82 (0.03)
Change the Width of an Existing Median	Rural	Four Lane	Full access control	See table 13-12 in HSM
			Partial or no access control	See table 13-13 in HSM
Increase the Distance to Roadside Features	Rural	Two Lane and Freeways	3.3- to 16.7-ft offset	0.78 (0.02)
			16.7- to 30.0-ft offset	0.56 (0.01)
Reduce Roadside Hazard Rating	Rural	Two Lane	$CMF = \frac{e^{(-0.6869+0.0668*RHR)}}{e^{(-0.4865)}}$	See figure 13-8 in 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 in HSM
Improve Superelevation	Rural	Two lane	Improve superelevation variance < 0.01	1.00 (N/A)
			Improve SV 0.01≤ SV < 0.02	1.00 + 6 (SV – 0.01)
			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)

CMF = Crash modification factor
HSM = *Highway Safety Manual*
N/A = Not applicable

APPENDIX B. CATALOG OF TRAFFIC ENGINEERING TREATMENTS

The technical information sheets in this appendix describe traffic engineering treatments shown to affect speed or speeding-related crashes on rural or suburban roadways. They describe each treatment and, along with a photograph, show the deployment condition. Design and construction specifications, safety and operational effectiveness, and cost are also described for each treatment. For ease of reference the treatments are presented in the following order:

- Speed table.
- Lateral shift.
- Speed kidney.
- Tubular channelizers.
- Transverse rumble strips.
- Longitudinal rumble strips.
- Converging chevron marking pattern.
- Transverse markings.
- Optical speed bars.
- Speed limit pavement legend.
- Enhanced speed limit legend with colored surfacing.
- SLOW pavement legend.
- Zigzag pavement markings.
- Painted medians.
- Add shoulder markings to narrow lane.
- Add centerline and edge line.
- Speed feedback sign.
- Speed-activated warning sign.
- Speed-activated speed limit reminder sign.
- Variable speed limit sign.
- Transverse pavement markings with speed feedback sign.
- Advisory speed limit sign.
- Red border speed limit sign.
- One-direction large arrow (W1-6) sign.
- Flashers to existing curve warning sign.
- Flags to existing curve warning sign.
- Combination horizontal alignment/advisory sign.
- Chevron sign.
- Curve warning sign and chevron sign and flashing beacons.
- Curve warning sign and chevron sign.
- Delineator post.
- Median barriers.
- Roadside barriers.
- Lane widening.
- Two-way left-turn lane.
- Additional lane (or road duplication).
- Paving shoulders.
- Road surface upgrades.
- Gateway treatment.

SPEED TABLE

Source: FHWA

Figure 141. Photo. Speed table.[53]

Description

Speed tables are midblock traffic calming devices that raise the entire wheelbase of a vehicle to reduce its traffic speed. Where applied, speed tables may be designed as raised midblock crossings, often in conjunction with curb extensions.[99]

Design Details

Speed tables are longer than speed humps and flat-topped, with a typical height of 3 to 3.5 inches and a length of 22 ft. Vehicle operating speeds for streets with speed tables range from 25 to 45 mph.[99]

Safety Effectiveness

No published safety evaluation for speed tables was identified.

Speed Reduction Effectiveness

Speed tables have been shown to reduce 85th percentile speeds by 4 mph (-14 percent) in small, rural towns and 9 mph (-24 percent) on rural, residential streets.[52,54]

Cost

The cost to construct a speed table ranges from $2,000 to $15,000.[100]

LATERAL SHIFT

Source: City of Sparks Public Works: Traffic Division, Reno, NV

Figure 142. Photo. Lateral shift.[101]

Description

A lateral shift is a curb extension that shifts travel lanes to one side of the road for an extended distance and then back to the other side.[102]

Design Details

A lateral shift generally includes a center island with curbing, which prohibits vehicles from entering the opposing lane. They may also incorporate landscaping the in center island.[101] The curb can be constructed from different materials, such as concrete, granite, or asphalt, depending on the desired appearance.

Safety Effectiveness

No published safety evaluation for a lateral shift was found.

Speed Reduction Effectiveness

Lateral shifts from the installation of raised traffic islands have been shown to reduce 85th-percentile speeds by 11 mph (-25 percent).[102]

Cost

The cost to construct a lateral shift can vary significantly, which is mainly dependent on the type of material (i.e., concrete or granite), size of the offset, and the length of the transition.[101]

SPEED KIDNEY

Source: Alfredo Garcia

Figure 143. Photo. Speed kidney.[103]

Description

A speed kidney is a longitudinal speed bump that takes the shape of a kidney that lowers vehicle speeds by forcing the driver to choose one of two slow vehicle paths.[103]

Design Details

The design consists of a curved speed bump in the middle-right of the travel lane and is complemented by an oval speed bump with the centerline as the long axis. This design forces the driver to take one of two vehicle trajectories. One is to continue in a straight line, causing lower speeds through mounting the speed bump with either one or two wheels. The other is following the curved path to avoid mounting the bump. This curved path leads to lower speeds as well.[103]

Safety Effectiveness

No published safety evaluation for this treatment was found.

Speed Reduction Effectiveness

The safety kidney has been found to reduce 85th percentile speeds by 11.2 mph in urban and rural areas.[103]

Cost

Costs are similar to those of speed humps and speed tables and are between $2,000 and $15,000.[103,100]

TUBULAR CHANNELIZERS

Source: FHWA

Figure 144. Photo. Tubular channelizers.(53)

Description

Tubular channelizers are 3-ft tall yellow markers placed on either side of the centerline to create an island. These can reduce lane width.(53)

Design Details

The 3-ft tall channelizers, mounted on either side of the centerline, are spaced at 4 ft within the tapers, while the rest of the markers are spaced at 8 ft. A speed limit sign can be mounted to indicate the desired speed for this portion of the roadway.(53)

Safety Effectiveness

No published safety evaluation for this treatment was found.

Speed Reduction Effectiveness

Channelizers have been shown to reduce 85th percentile speeds by 1 mph on main rural roads.(53)

Cost

The cost to construct tubular channelizers is estimated to be between $5,000 and $12,000. The reason for the high cost is that the tubes are often struck by vehicles and must be replaced.(53)

TRANSVERSE RUMBLE STRIPS

Source: FHWA

Figure 145. Photo. Transverse rumble strips.[104]

Description

Transverse rumble strips are raised or grooved patterns installed perpendicular to the direction of travel on the roadway travel lane or shoulder pavements.[105] Typically installed on rural roadways that have low volume and infrequent traffic control devices, rumble strips provide an audible and tactile warning of a downstream decision point.[106] They are different than centerline and edge-line rumble strips, which are located off the travel lanes.

Design Details

Designs for transverse rumble strips can vary greatly from site to site. Grooved rumble strips are typically 0.5 inches deep and 4 inches wide. Raised rumble strips typically range from 0.38 to 0.75 inches high and 4 inches wide.[107] They can either be placed as continuously or intermittently spaced sets of strips; spacing can vary based on operating speeds.[108]

Safety Effectiveness

Elvik and Vaa estimated the following CMFs for transverse rumble strips[109]:

- 0.66 for all crash types of all severities on urban and suburban roads.
- 0.64 for all crash types of serious and minor injuries on urban and suburban roads.

Effectiveness to Reduce Speed

Transverse rumble strips are shown to reduce speeds between 0.6 and 2.0 mph on rural highways.[105,106]

Cost

The cost estimate to install transverse rumble strips ranged from $0.10 to $0.60 per linear ft. The cost varies based on the length of the project, whether the surface is concrete or asphalt, and if they are installed as a standalone project or part of a larger project.[110]

LONGITUDINAL RUMBLE STRIPS

Source: FHWA

Figure 146. Photo. Longitudinal rumble strips.[111]

Description

Longitudinal rumble strips are raised or grooved patterns on the inside edge of the normal travel lane. These rumble strips have the ability to narrow effective lane width.[111]

Design Details

Longitudinal rumble strips can be placed on both the centerline and shoulder. Typical dimensions of rumble strips are 7 by 16 by ⅝ inches. The spacing is 12 inches center to center. The rumble strips should be milled, but can also be rolled in.[111]

Safety Effectiveness

Numerous safety studies prove the effectiveness of longitudinal rumble strips. Some studies have found that the treatment can reduce injury crashes by 14 percent, severe crashes by 18 percent, fatal and serious crashes by 67 percent, and overall crashes by 20 percent. (See references 112 through 115.)

Speed Reduction Effectiveness

Longitudinal rumble strips have been found to reduce 85th percentile speeds by 4.5 mph.[63]

Cost

The cost to install rumble strips ranges between \$500 and \$3,000 per mi).[111]

CONVERGING CHEVRON MARKING PATTERN

This pattern is not MUTCD compliant and requires FHWA experimental permission.

Source: Iowa State University

Figure 147. Photo. Converging chevron marking pattern.[52]

Description

The converging chevron pavement marking pattern involves installing a series of white chevrons on the road surface. The spacing width of the chevrons and the space between them decreases as the driver travels through the pattern. This pattern creates the illusion that the vehicle is traveling faster than the vehicle's actual speed and that the road is narrowing, which causes the driver to slow down.

Design Details

Use the decreasing velocity linear equation shown in to determine spacing between each pair of bars:

$$L = v_1 * t_b + \frac{(v_1^2 - v_2^2)}{2a}$$

Figure 148. Equation. Decreasing velocity linear equation

Where:

L = distance between successive pair of transverse bar pairs $pair_1$ and $pair_2$ (ft)
v_1 = speed at pair 1 (ft/s) (speed at the first pair is the transition zone speed; speed at the last pair is the entrance posted speed limit)
v_2 = speed at pair 2
t_b = perception reaction time (0.5 s)
a = deceleration rate (ft/s^2)

Safety Effectiveness

Converging chevron pavement markings were shown to reduce crashes by as much as 43 percent.[116]

Speed Reduction Effectiveness

Studies of the effectiveness of converging chevron pavement marking patterns show the potential to reduce 85th percentile speeds by 11 to 24 percent.[116] An FHWA study reported a 3-mph reduction in 85th percentile speeds.[53]

Cost

The cost to implement chevron pavement markings can range between $100 and $200 per marking.[117] Maintenance of converging chevrons involves regular repainting of the markings to maintain visibility.[53]

TRANSVERSE MARKINGS

Source: Virginia Center for Transportation Innovation and Research

Figure 149. Photo. Transverse markings.[75]

Description

Transverse markings are a series of white transverse bars—either flat or raised—placed across the center of the lane and spaced progressively closer together to create the illusion that driver speed is increasing.[118]

Design Details

The design for transverse markings can vary from site to site. Meyer tested three different layouts of transverse markings.[119] One involved installing 20 bars of consistent width and spacing intended to warn drivers. The next layout had varied width and spacing to create the illusion of traveling faster. The third, for work zones, installed four sets of six bars.

Safety Effectiveness

Although there are estimates of CMFs for transverse markings on roundabout approaches, no high-quality safety evaluation of transverse markings was found.

Speed Reduction Effectiveness

Transverse marking have been shown to marginally reduce 85th percentile speeds, 0.2 mph (0.3) on rural, horizontal curves.[118] Their application on rural highways has shown a decrease in 85th percentile speeds by the following amounts:

- 1.4 and 3.9 mph.[120]
- 3 to 10 mph.[75]
- 4 mph (-11 percent) in work zones.[119]

Cost

The cost to construct or install transverse pavement marking varies based on the material (i.e., paint or thermoplastics), the number of transverse bars, and the width of each bar. Traffic paint can be as low as $0.10/ft, and the price of wet reflective tape can range between $1 and $2/ft.[121] Generally, the cost of a single installation will be less than $2,500.[53] The maintenance needed for transverse markings is regular painting of the markings to maintain visibility.[53]

OPTICAL SPEED BARS

Source: Virginia Center for Transportation Innovation and Research

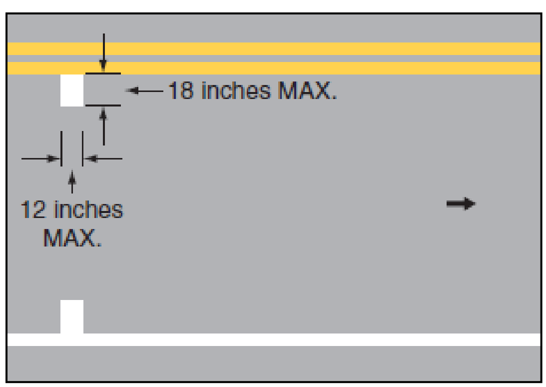
Figure 150. Photo. Optical speed bars.[(75)]

Description

OSBs are transverse markings with progressively reduced spacing installed on the roadway within a lane in a pattern to give drivers the impression that their speed is increasing. OSB can be used on the whole roadway (transverse) or just on the edges (peripheral).

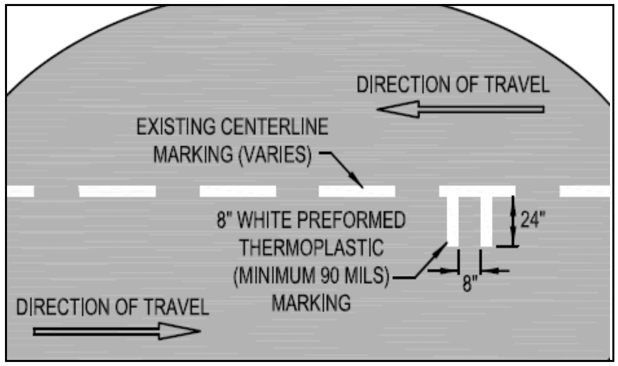
Design Details

Peripheral optical bars, 12 by 18 inches in size, must be installed on both sides of the lane perpendicular to the centerline, edge line, or lane line. Peripheral optical bars cannot be used in lanes that do not have a longitudinal line (centerline, edge line, or lane line) on both sides of the lane (in accordance with 2009 MUTCD, Part 3B. 22).[(122)] The spacing of the bars decreases to maintain a 4-bar/s spacing. The length of the segment also needs to provide a minimum of 4 s of driving time within the OSBs.[(104)] Mohave County, AZ, used a variation of the MUTCD design . In this design, two 24- by 8-inch bars, spaced 8 inches apart, were placed perpendicular to the centerline.



Source: FHWA

Figure 151. Illustration. Peripheral Design (2009 MUTCD, Part 3B. 22).[(122)]



Source: Mohave County Public Works, Arizona

Figure 152. Illustration. Peripheral Design Variation, Mohave County, AZ.

A variation to the typical transverse bars was used by Iowa.(123) This design consisted of three horizontal bars spaced at intervals so that drivers can position their vehicles within the wheel paths.

Spacing between each pair of bars is determined using a decreasing velocity linear equation.(76)

Source: Iowa State University

Figure 153. Photo. OSB Variation, Iowa State University.(124)

Safety Effectiveness

No published safety evaluation for OSBs was found.

Speed Reduction Effectiveness

Peripheral Speed Bars have been shown to reduce 85th percentile speeds by the following amounts:

- 0 mph.(125)
- 0 to 2 mph.(55)
- 1 to 3 mph.(77,53,57)
- 4 to 5 mph—variation from MUTCD design.(58)

Cost

The cost to implement is approximately \$8/ft^2. The maintenance for transverse markings requires regular painting of the markings to maintain visibility.[(53)]

SPEED LIMIT PAVEMENT LEGEND

Source: Iowa State University

Figure 154. Photo. Speed limit pavement legend.[(126)]

Description

A speed limit pavement legend places pavement markings at regularly spaced intervals to remind drivers of the speed limit. The speed limit pavement legend can be placed as a standalone treatment, at the end of converging chevrons, or in combination with lane-narrowing measures.[(53)]

Design Details

No specific design details (i.e., width and height) were found for the speed limit pavement legend.

Safety Effectiveness

No published safety evaluation for speed limit pavement legends was found.

Speed Reduction Effectiveness

Studies indicate that speed limit pavement legends can reduce 85th percentile speeds by 1 mph (-1 percent) on rural, main roadways.[(53)]

Cost

The cost to install speed limit pavement legends varies based on how often the legend is applied but generally is less than $2,500. The maintenance needed for speed limit pavement legends is regular repainting of the legend to maintain visibility.[(53)]

ENHANCED SPEED LIMIT LEGEND WITH COLORED SURFACING

The colored outline is not MUTCD compliant and requires FHWA experimental permission.

Source: Iowa State University

Figure 155. Photo. Enhanced speed limit legend with colored surfacing.[123]

Description

An enhanced speed limit legend with colored surfacing involves the same treatment as the speed limit pavement legend, except the legend is surrounded by an additional colored box. The legends are spaced at regular intervals to remind drivers of the speed limit.[53]

Design Details

The red-colored surface was used to evaluate the effectiveness of enhanced speed limit legends. A large red rectangle (9.5 by 12 ft) was placed around the on-pavement speed limit markings, which were 35 mph at the test sites. The edge line was also widened to 8 inches along the treatment to enhance visibility.[53]

Safety Effectiveness

No published safety evaluation for enhanced speed limit legends with colored surfacing was found.

Speed Reduction Effectiveness

Studies show that enhanced speed limit legends with colored surfacing can reduce 85th-percentile speeds by 2 mph (-4 percent) on rural, main roadways.[53]

Cost

The cost to enhanced speed limit legends with colored surfacing varies based on how often the legend is applied, but generally is less than $2,500. The maintenance required is regular repainting of the legend to maintain visibility; the red background has been shown to fade quickly, so there may need to be an accelerated repainting cycle.[53]

SLOW PAVEMENT LEGEND

Source: Iowa State University

Figure 156. Photo. Slow pavement legend.[(52)]

Description

A pavement marking legend is installed in the travel lane to indicate SLOW—the driver should reduce speed.

Design Details

One study installed the SLOW pavement marking prior to a horizontal curve. The word SLOW was installed approximately 220 ft upstream of the PC of a horizontal curve. The marking consisted of the word SLOW in 8-ft-high white letters. There was also an 8-ft-high arrow installed above the word SLOW. In addition to these two markings, an 18-inch-wide white line was installed perpendicular to the road, at the beginning and end of the marking.[(127)]

Safety Effectiveness

No published safety evaluation for installing SLOW pavement legends was found.

Speed Reduction Effectiveness

SLOW pavement legends have been shown to increase 85th percentile speeds by 1 mph, but they have also been shown to decrease 85th percentile speeds by 2 mph.[(53,129)]

Cost

The cost to install a SLOW pavement legend is less than $2,500. The maintenance needed for the pavement legend is regular repainting to maintain visibility.[(53)]

ZIGZAG PAVEMENT MARKINGS

Zigzag pavement markings are not included in MUTCD.

Source: Virginia Center for Transportation Innovation and Research

Figure 157. Photo. Zigzag pavement markings.[(61)]

Description

A painted zigzag line in the center of the travel lane is used to indicate upcoming pedestrian crossings or horizontal curves.[(61)]

Design Details

The zigzag extends 500 ft from the crosswalk or beginning of the curve. The width of the painted line should be larger than the centerline and edge lines, meaning 6 inches should suffice for 4-inch edge and centerline markings. The lateral width of the zigzag is 4 ft, smaller than typical vehicle track widths to avoid paint wear from traffic. The length of one line in the zigzag is 12 ft. The zigzag should be painted white.[(61)]

Safety Effectiveness

No published safety evaluation for this treatment was found.

Speed Reduction Effectiveness

The zigzag markings have been found to reduce 85th percentile speeds by up to 1.3 mph in suburban areas.[(61)]

Cost

Initial costs for the pavement markings are estimated to be $2,850, including labor and materials. The total cost over 5 years should be expected to be $5,700, assuming reapplication of the lines in 2 to 3 years.[(61)]

PAINTED MEDIANS

Source: FHWA

Figure 158. Photo. Painted medians.[128]

Description

Typically used at intersections, painted medians are flush medians marked with painting that separate opposing traffic lanes.[122]

Design Details

Widths of painted medians vary depending on available right-of-way. Markings consist of edge lines and cross hatching and should be implemented in accordance with Section 3B.03 of the MUTCD.[122]

Safety Effectiveness

This treatment has been found to reduce fatal crashes on rural and urban roads by 20 percent.[129]

Speed Reduction Effectiveness

No published speed evaluation for this treatment was found.

Cost

Costs are typically between $10,000 and $30,000 per intersection, but they can range as high as $70,000.[130]

ADD SHOULDER MARKINGS TO NARROW LANE

Source: FHWA

Figure 159. Photo. Add shoulder markings to narrow lane.(53)

Description

Installing wide edge line and crosshatch markings creates a shoulder on both sides of the roadway to reduce lane widths.

Design Details

In the study that evaluated this treatment, an existing two-lane roadway that was 36 ft wide from curb to curb was reduced to single 10.5-ft lanes in both directions. This was accomplished by using wide edge line and crosshatching. The study suggested that it may be more effective to reduce the lane width even more, or it may be necessary to install physical barriers because there are no consequences for crossing the pavement markings.(53)

Safety Effectiveness

No published safety evaluation for adding shoulder markings to narrow lanes was found.

Speed Reduction Effectiveness

Adding shoulder markings to narrow lanes has had minimal effects on 85th percentile speeds, with an increase of 0.5 mph.(53)

Cost

Traffic paint can be as low as $0.10/ft and the price of wet reflective tape can range between $1 and $2/ft.(121) Generally, the cost will be less than $2,500.(53) The maintenance needed for shoulder markings is regular repainting of the markings to maintain visibility.(53)

ADD CENTERLINE AND EDGE LINE

Source: FHWA

Figure 160. Photo. Add centerline and edge line.[53]

Description

Installation of median and shoulder pavement markings reduces lane widths.

Design Details

The pavement markings can vary from site to site, based on the characteristics of each site and the intended purpose. One study added a 10-ft-wide painted center island and added a 6-inch edge line to separate the travel lane from the 8-ft parking lane. The lane widths decreased from approximately 16 to 11 ft in each direction).[53]

Safety Effectiveness

Studies show that adding centerline and edge line pavement markings can reduce fatal and injury crashes by 8.6 percent.[48] There are many CMFs for different edge line and shoulder treatments. Installing edge lines on tangents and curves has the following CMFs:[131]

- 0.921 for all crash types of all severities.
- 0.888 for all run-off-road crashes of all severities.

Speed Reduction Effectiveness

Adding a centerline and edge line have been shown to reduce 85th percentile speeds by the following amounts:

- 1 mph on rural, main roads.[53]
- 2 mph during the day on rural, two-lane roads.[132]
- 1 mph during the night on rural, two-lane roads.[132]
- 3.5 mph at rural, two-way, stop-controlled intersections.[48]

Cost

The cost to install centerline and edge line pavement markings can be as low as $0.10/ft for paint, but the price of wet reflective tape can range between $1 and $2/ft.[121] Generally, the cost will be less than $2,500.[53] The maintenance needed for shoulder markings is regular painting of the markings to maintain visibility.[53]

SPEED FEEDBACK SIGN

Source: Iowa State University

Figure 161. Photo. Speed feedback sign.(126)

Description

A speed feedback sign is a dynamic sign that displays the speed of approaching vehicles.(53)

Design Details

Speed feedback signs vary based on the site. Most signs display the text, YOUR SPEED along with the measured speed of the approaching vehicle.(53)

Safety Effectiveness

No published safety evaluation for speed feedback signs was found.

Speed Reduction Effectiveness

Studies indicate that speed feedback signs can reduce 85th percentile speeds by the following amounts:

- 7 mph.(53)
- 2 mph.(62)
- 3 mph.(63)
- 4 mph.(64)
- 2 mph.(65)
- 6.3 mph.(66)
- 2 to 3 mph.(67)

Cost

The cost to install a speed feedback sign can range from \$5,000 to \$12,000.(53)

SPEED-ACTIVATED WARNING SIGN

Source: Clemson University

Figure 162. Photo. Speed-activated warning sign. (133)

Description

A speed-activated warning sign consists of a fixed-message, speed-activated sign that triggers a flashing beacon when a predetermined speed threshold is exceeded. The speed-activated sign is a 4- by 4-ft corrugated plastic reflective sign with 6-inch lettering reading "YOU ARE SPEEDING IF FLASHING." To increase the speed-activated sign's visibility, two 1- by 1-ft orange plastic flags were added.

Design Details

The design of speed-activated warning signs can vary from one site to another, depending on the intended purpose of the sign. The sign shown above and the one used by Mattox et al. was a 4- by 4-ft corrugated plastic reflective sign with 6-in lettering reading YOU ARE SPEEDING IF FLASHING.(131) A beacon atop the sign flashes when a vehicle exceeds a certain threshold.(133) This is a temporary sign; however, there are also permanent speed-activated signs.

Safety Effectiveness

No published safety evaluation for speed-activated warning signs was found.

Speed Reduction Effectiveness

Speed-activated warning signs are shown to reduce 85th percentile speeds by the following amounts:

- 1.6 to 4.7 mph on multilane highways.(133)
- 4 mph on rural, four-lane-divided highways.(134)
- 1 mph on rural curves on Interstates.(135)

Cost

The cost to install the sign used by Mattox et al. was approximately $1,500.(133) Other, more permanent installations can cost between $5,000 and $12,000.(53)

SPEED-ACTIVATED SPEED LIMIT REMINDER SIGN

Source: SWARCO Traffic Ltd

Figure 163. Photo. Speed-activated speed limit reminder sign.

Description

A speed-activated-speed limit reminder sign is an electronic speed-limit sign activated by an approaching vehicle.

Design Details

The vehicle-actuated fiber-optic 30-mph sign was installed 624 meters (680 yd) prior to a 20-mph zone.[136]

Safety Effectiveness

No published safety evaluation for speed-activated speed limit reminder signs was found.

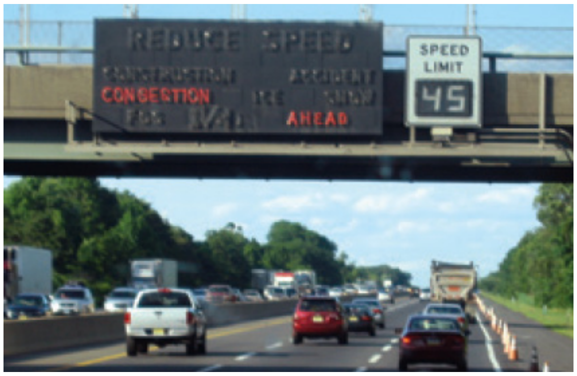
Speed Reduction Effectiveness

Studies show that speed-activated speed limit reminder signs can reduce 85th percentile speeds by 5 mph on major roads.[136]

Cost

The cost to install a speed-activated speed limit reminder sign is comparable to other electronic signs with radar, which can range between $2,000 and $12,000.[53]

VARIABLE SPEED LIMIT SIGN



Source: FHWA

Figure 164. Photo. Variable speed limit sign.[(137)]

Description

A variable speed limit sign is an electronic message sign that displays dynamic information related to the speed limit. These signs can indicate a reduced speed limit during wet or low-light weather conditions, which may improve safety by decreasing the risks associated with vehicles traveling at speeds higher than appropriate for the conditions.[(137)]

Design Details

Variable speed limit signs can be used in place of static speed limit signs placed to the right of the shoulder, or they can be mounted over the individual lanes of a multilane highway and individual lanes can have varying speed limits. A variable speed limit for wet conditions is justified when adverse weather conditions cause traffic problems, when there is a higher crash rate than expected for similar segments, the regular occurring safe speed limit is at least 10 mph less than the normal speed limit, or there are conditions when stopping distance exceeds available sight distance.

Safety Effectiveness

The following CMF for variable speed limit signs has been estimated:

- 0.92 for all crash types of all severities in urban areas (four star).[(138)]

Speed Reduction Effectiveness

Variable speed limit signs have been shown to reduce 85th percentile speeds by the following amounts:

- 4.7 to 8 mph.[(139)]
- 5 mph.[(140)]
- 0.47 to 0.75 mph per 1-mph reduction in the PSL.[(141)]

Cost

The cost to install a single variable speed limit sign is between $5,000 and $12,000, similar to speed feedback signs.[53] The cost to install variable speed limits on a multilane highway is significantly higher than the cost of the overhead structure.

TRANSVERSE PAVEMENT MARKINGS WITH SPEED FEEDBACK SIGN

Source: FHWA

Figure 165. Photo. Transverse pavement markings with speed feedback sign.[53]

Description

Transverse pavement markings are placed at the entrance to a community. Speed feedback signs were placed immediately downstream of the transverse pavement markings.[53]

Design Details

The markings are 12 inches wide by 18 inches long (perpendicular to roadway edge). Spacing between the bars decreases approaching the neighborhood, which can cause the driver to slow down because the visual sense is one of speeding up. A static speed feedback sign is placed at the end of the markings. These signs display YOUR SPEED and show the current vehicle speed.[53]

Safety Effectiveness

No published safety evaluation for this treatment was found.

Speed Reduction Effectiveness

This combination of treatments has been found to reduce 85th percentile speeds by 4 mph.[53]

Cost

The cost of this treatment is estimated to range between $5,000 and $12,000, with regular painting being the most significant maintenance concern.[53]

ADVISORY SPEED LIMIT SIGN

Source: FHWA

Figure 166. Photo. Advisory speed limit sign.[122]

Description

This treatment entails installing a static yellow advisory speed limit sign to provide drivers with advisory speeds for less than ideal roadway conditions.[122]

Design Details

Use the sign as a supplement to another advisory sign, such as for a turn or horizontal curve. Size is typically either 18 by18 inches or 24 by 24 inches. Install the sign following the guidelines set forth for the advisory speed limit sign (W13-1P) and the following signs: W13-2, W13-3, W13-6, and W13-7, in the MUTCD.[122]

Safety Effectiveness

Published CMFs indicate that advisory speed limit signs installed in combination with horizontal alignment warning signs can affect safety as follows[109]:

- 13-percent reduction (CMF = 0.87) in injury crashes.
- 29-percent reduction (CMF = 0.71) in PDO crashes.

Speed Reduction Effectiveness

Installing an advisory speed limit sign has been found to reduce 85th percentile speeds by 15 percent.[142]

Cost

A 30-inch advisory speed limit sign with engineer-grade reflective sheeting costs approximately $50.[143]

RED BORDER SPEED LIMIT SIGN

Colored border is not MUTCD compliant and requires FHWA experimental permission.

Source: Texas Transportation Institute

Figure 167. Photo. Red border speed limit sign.[144]

Description

A red border speed limit sign is a standard speed limit sign with an added red border. The red border is designed to increase the visibility of the sign, and drivers may recognize an increased sense of importance because of the enhanced red border.[59]

Design Details

Two different red border signs have been used. The first is a standard speed limit sign (R2-1) that adds 6 inches to the height and width that allows for a 3-inch red border to be added around the existing speed limit sign. The second red border sign removed the black border from the standard speed limit sign and replaced it with a 1-inch red border along with the 3-inch border from the first red border sign. This sign had a 4-inch red border.[58]

Safety Effectiveness

No published safety evaluation for installing red border speed limit signs was found.

Speed Reduction Effectiveness

Red border speed limit signs have been shown to reduce 85th percentile speeds by 3 mph on two-lane highways.

Cost

The cost to install a red border speed limit sign is similar to a standard speed limit sign plus the cost to modify the sign.

ONE-DIRECTION LARGE ARROW (W1-6) SIGN

Source: KLS Engineering, LLC

Figure 168. Photo. One-direction large arrow (W1-6) sign.

Description

A one-direction large arrow sign (W1-6) is placed on a horizontal curve to alert drivers that they are approaching a horizontal curve and its direction.

Design Details

A one-direction large arrow sign (W1-6) is designed in accordance with the MUTCD. For a single lane or multilane road, a W1-6 sign is 48 by 24 inches. On an expressway or freeway, a W1-6 sign is 60 by 30 inches. A one-direction large arrow sign is placed on the outside of a curve at a right angle to oncoming traffic. As with any warning sign, a W1-6 sign has a yellow background with a black arrow.(122)

Safety Effectiveness

No published safety evaluation for installing a one-direction large arrow was found.

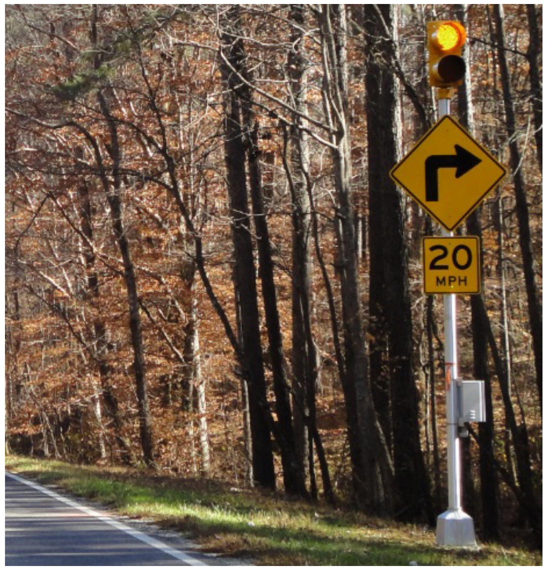
Speed Reduction Effectiveness

One-direction large arrows have been shown to have no effect (0 mph speed reduction) on 85th percentile speeds of vehicles on rural, horizontal curves.(118)

Cost

The cost to install a one-direction large arrow sign is less than $500.(145)

ADD FLASHERS TO EXISTING CURVE WARNING SIGN



Source: KLS Engineering, LLC

Figure 169. Photo. Add flashers to existing curve warning sign.

Description

Six-inch flashing amber lights are added to an existing curve warning sign.(118)

Design Details

Two flashing amber lights are added to an existing curve warning sign.

Safety Effectiveness

No published safety evaluation for adding flashers to an existing curve warning signs was found.

Speed Reduction Effectiveness

Adding flashers to an existing curve warning sign has been shown to increase 85th percentile speeds by 1 mph.(118)

Cost

The cost to add flashing lights to an existing sign is less than $500.(145)

ADD FLAGS TO EXISTING CURVE WARNING SIGN

Source: University of Kentucky

Figure 170. Photo. Curve warning sign with flags.[118]

Description

Red flags are added to an existing curve warning sign to attract driver attention.

Design Details

Two red flags are added to an existing curve warning sign.

Safety Effectiveness

No published safety evaluation for adding flags to an existing curve warning signs was found.

Speed Reduction Effectiveness

Adding flags to an existing curve warning sign has been shown to reduce 85th percentile speeds by 0.3 mph.[118]

Cost

The cost to add flags to an existing sign is less than $500.

COMBINATION HORIZONTAL ALIGNMENT/ADVISORY SIGN

Source: FHWA

Figure 171. Photo. Combination horizontal alignment/advisory sign.[122]

Description

A sign containing both a curve or turn ahead and advisory speed is installed prior to a horizontal curve.

Design Details

A combination Turn/Advisory Speed (W1-1a) and a combination Curve/Advisory Speed (W1-2a) are designed in accordance with the MUTCD. These signs can used to supplement the advanced horizontal alignment warning sign and advisory speed plaque, but they should not be used alone or used as a substitute for a horizontal alignment warning sign and advisory speed plaque. Install the W1-1a and W1-2a sign at the beginning (PC) of the turn or curve, if installed. For a single lane or multilane road, W1-1a and W1-2a signs are 36 by 36 inches. On an expressway or freeway, W1-1a and W1-2a signs are 48 by 48 inches. As with any warning sign, W1-1a and W1-2a signs have a yellow background and black arrows/text.[122]

Safety Effectiveness

The following CMFs for combination Turn/Advisory Speed and combination Curve/Advisory Speed signs have been estimated:[109]

- 0.87 for all crash types that are serious and minor injuries.
- 0.71 for all crash types that are PDO.

Speed Reduction Effectiveness:

Installing a combination Turn/Advisory Speed or combination Curve/Advisory Speed sign has been shown to increase 85th percentile speeds by 0.2 mph.[146]

Cost

The cost to install a combination Turn/Advisory Speed or combination Curve/Advisory Speed sign is less than $500.[145]

CHEVRON SIGN

Source: Iowa State University

Figure 172. Photo. Chevron sign.[52]

Description

Installing chevron alignment signs (W1-8) on a horizontal curve provides additional guidance and emphasis for a change in horizontal alignment.

Design Details

Install chevron alignment signs in accordance with the MUTCD. For a single lane or multilane road, a W1-8 sign is 18 by 24 inches. On an expressway, a W1-8 sign is 30 by 36 inches. On an expressway, a W1-8 sign is 36 by 48 inches. Install chevron alignment signs at a minimum height of 4 ft. The spacing of the signs depends on the curve radius and is guided by Table 2C-6 in the MUTCD. As with any warning sign, a W1-8 sign has a yellow background with a black arrow.[122]

Safety Effectiveness

The following CMFs for chevrons have been estimated (all four-star):[147]

- 0.96 for non-intersection crashes of all severities.
- 0.94 for head on, non-intersection, run-off-road, and sideswipe crashes of all severities.
- 0.84 for non-intersection, fatal, serious injury, and minor injury crashes.
- 0.75 for nighttime, non-intersection crashes of all severities.
- 0.78 for head on, nighttime, non-intersection, run off road, and sideswipe crashes of all severities.

Speed Reduction Effectiveness

Chevron alignment signs have been shown to reduce 85th percentile speeds on rural horizontal curves by the following amounts:

- 0.7 mph.[118]

- 2.4 to 5.28 km/h.[148]
- 1.28 mph.[149]

Cost

The cost to install a chevron alignment sign is less than $500.[145]

CURVE WARNING SIGN AND CHEVRON SIGN AND FLASHING BEACONS

Source: Michigan Department of Transportation

Figure 173. Curve warning sign and chevron sign and flashing beacons.

Description

Curve warnings signs are placed on the approach to a horizontal curve. These are supplemented with flashing beacons to attract the driver's attention and chevrons to delineate the curve.(104)

Design Details

Refer to MUTCD guidelines for Curve Warning Signs (W1-1, W1-2) and Chevron Alignment Signs (W1-8). Also refer to MUTCD, Chapter 4L, for proper use of flashing warning beacons.(104,122)

Safety Effectiveness

There are many safety benefits of this signage combination, including numerous crash modification factors. Some published CMFs indicate a reduction of 40 percent in all types of crashes on rural roads. The treatment was also found to reduce all types of crashes under wet conditions by 47 percent.(150)

Speed Reduction Effectiveness

No published speed evaluation for this treatment was found.

Cost

Costs for this treatment vary depending on the number of chevron signs (W1-8) installed—10 chevron signs typically cost $500. The cost of the curve warning sign (W1-2) can vary based on size and quality but should not exceed $500. A solar-powered yellow beacon should cost about $2,200, more if solar power is not used.(104,117)

CURVE WARNING SIGN AND CHEVRON SIGN

Source: Tom Welch

Figure 174. Photo. Curve warning sign and chevron sign.

Description

Curve warnings signs are placed on the approach to a horizontal curve. These are supplemented with chevrons placed along the curve.(104)

Design Details

Refer to 2009 MUTCD Section 2C.10 for Curve Warning Signs (W1-1, W1-2) and Chevron Alignment Signs (W1-8) guidelines.

Safety Effectiveness

There are many safety benefits of this signage combination. In general, this treatment has a CMF of 0.60, indicating a 40-percent reduction in all crashes on rural roads. This combination of signage can also reduce nighttime crashes on rural horizontal curves by 40.8 percent (CMF = 0.592).(150)

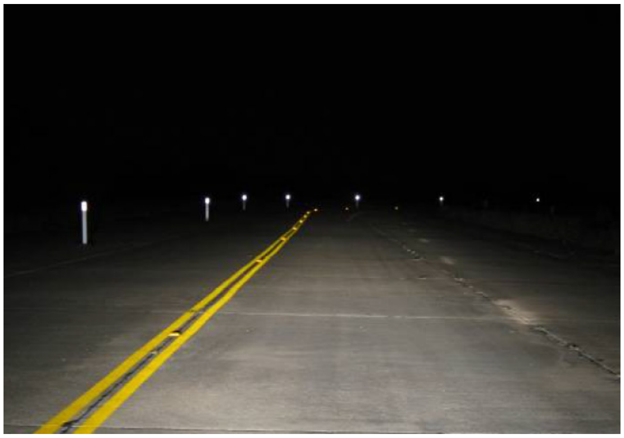
Speed Reduction Effectiveness

No published speed evaluation for this treatment was found.

Cost

Costs for this treatment vary depending on the number of chevron signs (W1-8) installed. Ten chevron signs typically cost $500. The curve warning sign (W1-1, W1-2) can vary based on size and quality. In total, the cost (including the chevrons) should be less than $1,000.(104,117)

DELINEATOR POST



Source: Texas Transportation Institute

Figure 175. Photo. Delineator post.[151]

Description

Delineator posts are retroreflective devices mounted above the roadway surface and along the side of the roadway in a series to emphasis the roadway alignment and guide drivers. They provide the benefit that they remain visible when the road is wet.[122]

Design Details

The retroreflective element of a delineator has a minimum 3-inch height. On mainline tangent sections, the distance between delineators should be between 200 and 530 ft. The distance between delineators on horizontal curves is determined by the radius of the curve, which Table 3F-1 in the MUTCD shows. Delineators are typically placed between 2 and 8 ft beyond the shoulder edge.[122]

Safety Effectiveness

The following CMFs are estimated for delineator posts:[109]

- 1.04 for all crash types for serious and minor injuries in rural areas.
- 1.05 for all crash types for property damage only that are in rural areas (PDO).
- 0.55 for all crash types for serious and minor injuries.

There are also reports of a 30-percent reduction in crash after installing delineator posts.[38]

Speed Reduction Effectiveness

Delineator posts have been shown to slightly increase 85th percentile speeds by 0.5 mph on rural, horizontal curves.[118] They have also been shown to reduce 85th percentile speeds by 7 percent.[142]

Cost

The cost to install a delineator post is approximately \$30−\$50 per delineator.(145)

MEDIAN BARRIERS

Source: KLS Engineering, LLC

Figure 176. Photo. Median barriers.

Description

A median barrier is a physical barrier used to separate opposing traffic.(152)

Design Details

Many types of barriers can be used in this setting, including tubular channelizers, cable barriers, guiderail, and concrete barriers. These barriers are placed along the centerline of the roadway to separate traffic. Refer to AASHTO roadside design guidance for complete specific design details.(153)

Safety Effectiveness

Median barriers have been found to reduce fatal crashes by 50 percent in both urban and rural settings.(152) There are numerous CMFs regarding median barriers that depend on the type of barrier used. In general, simply adding a median barrier reduces fatal crashes on rural roadways by 43 percent (CMF = 0.57) and injury crashes by 30 percent (CMF = 0.70).(109)

Speed Reduction Effectiveness

No published speed evaluation for this treatment has been found.

Cost

Costs vary based on barrier used. Typical costs range from $50,000 to $300,000 for cable barriers, $83,000 to $600,000 for guardrail, and $120,000 to $2.7 million for concrete barriers.(153)

ROADSIDE BARRIERS

Source: KLS Engineering, LLC

Figure 177. Photo. Roadside barriers.

Description

Roadside barriers are placed along the side of the road to safely redirect vehicles from roadside hazards.(154)

Design Details

Multiple types of roadside barriers can be used. These include flexible barriers such as cable barriers, semi-rigid barriers such as guardrail, and rigid barriers such as concrete barriers. Design should conform to AASHTO design guidance.(28)

Safety Effectiveness

This treatment is found to reduce crashes by 40 percent in both urban and rural areas.(155) The following CMFs related to roadside barriers have been published:(109)

- 0.53 (47-percent reduction) for minor and serious injury run-off-road crashes.
- 0.56 (44-percent reduction) for fatal run-off-road crashes.

Speed Reduction Effectiveness

No published speed evaluation for this treatment was found.

Cost

Costs vary based on barrier used. Typical costs range from $50,000 to $300,000 for cable barriers, $83,000 to $600,000 for guardrail, and $120,000 to $2.7 million for concrete barriers.(153)

LANE WIDENING

Source: FHWA

Figure 178. Photo. Lane widening.[48]

Description

Lane widening, as indicated, increased the width of a travel lane.

Design Details

Lanes can be widened by adding paved width to the cross section.

Safety Effectiveness

The safety effectiveness of lane widening is based on original and new lane width. Research shows that in urban and rural environments, increases in lane width from 9 to 10 ft result in a 13-percent reduction in crashes. A change from 10 to 11 ft results in a 19-percent reduction in crashes. Finally, an increase from 11 to 12 ft only results in a 5-percent reduction.[129] Published CMFs exist but are not listed here because they are contingent on original and modified lane width.

Speed Reduction Effectiveness

No published speed evaluation for this treatment was found.

Cost

Costs vary significantly based on right-of-way acquisition, pavement design, etc. Refer to the local transportation agency for more accurate cost estimation for road construction and modification in the area.

TWO-WAY LEFT-TURN LANE

Source: FHWA

Figure 179. Photo. Two-way left-turn lane.[156]

Description

A TWLTL is a wide, painted center turning lane that also functions as a median.[156]

Design Details

A TWLTL ranges between 10 and 16 ft wide. The edges are marked by a solid yellow line on the side of traffic and a broken yellow line on the inside. Opposing left-turn arrows are also painted at regular intervals along the length of the lane.[28]

Safety Effectiveness

TWLTLs have been found to reduce crashes on rural two-lane roads by 29 percent.[156] The following are estimated CMFs for TWLTL:[157]

- 0.69 (31-percent reduction) in all crashes on approaches to unsignalized three-legged intersections.
- 0.66 (34-percent reduction) in all crashes on approaches to unsignalized four-legged intersections.

Speed Reduction Effectiveness

TWLTLs have been found to reduce 85th percentile speeds by 12.6 km/h (7.8 mph) on rural and suburban roads.[71]

Cost

Costs vary depending on State, with Arkansas, North Carolina, and California finding costs between $425,000 and $500,000 per mi to build, while the cost in Illinois was $1.78 million per mi. Maintenance costs are negligible compared with these prices.[156]

ADDITIONAL LANE (OR ROAD DUPLICATION)

Source: KLS Engineering, LLC

Figure 180. Photo. Additional lane.

Description

Adding an extra lane in one (or both) directions to provide an opportunity for fast-moving vehicles to pass slower moving ones. These can be classified as climbing lanes or passing lanes.(28)

Design Details

The additional travel lane should have the same characteristics of the existing travel lanes. If the intended function of the travel lane is as a truck climbing lane, the design should be based on guidelines in the *AASHTO Green Book*. Similarly, the *Green Book* can provide guidance if the lane's intended function is as a passing lane.(28)

Safety Effectiveness

This treatment has been found to reduce crashes on rural roads by between 25 and 50 percent.(152,129)

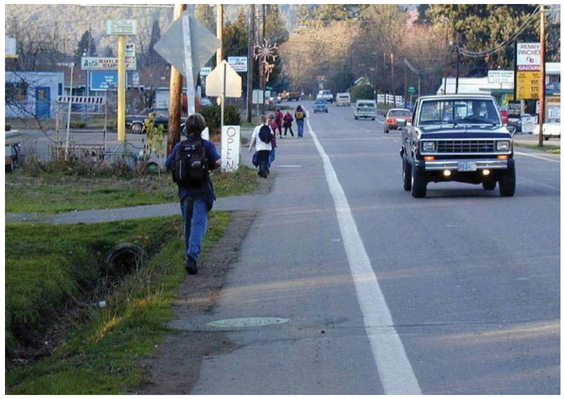
Speed Reduction Effectiveness

No published speed evaluation for this treatment was found.

Cost

Costs vary significantly based on right-of-way acquisition, pavement design, etc. Refer to the local transportation agency for accurate cost estimation for road construction in the area.

PAVING SHOULDERS



Source: FHWA

Figure 181. Photo. Paving shoulder.[158]

Description

The addition of a paved shoulder provides a paved surface for an errant vehicle to recover and return to the roadway.[155]

Design Details

The preferred width of a paved shoulder is between 6 and 8 ft at a slope steeper than the travel lane. For example, if a travel lane has a 2-percent cross slope, then the shoulder should be at least 4 percent. Delineate the shoulder with a painted edge line.[28]

Safety Effectiveness

A paved shoulder has been found to reduce crashes on urban and rural roadways by 30 percent.[155] Published CMFs exist for this treatment. These factors vary based on the previous shoulder before paving and the dimensions of the new shoulder.

Speed Reduction Effectiveness

No published speed evaluation for this treatment was found.

Cost

The cost to construct a paved shoulder ranges from \$18.47/yd^2 to \$111.19/yd^2. This price range is so diverse because the cost is based on materials used and pavement depth.[159]

ROAD SURFACE UPGRADES

Source: KLS Engineering, LLC

Figure 182. Photo. Road surface.

Description

Road surface upgrades are pavement resurfacing projects, mainly performed to support the structural strength of the roadway.(160)

Design Details

Designs for roadway resurfacing projects are context sensitive. Depth and material should be based on traffic loading, environment, local and state agency guidelines, etc. Refer to the local transportation agency for specific pavement resurfacing guidelines.

Safety Effectiveness

Multiple studies have quantified the safety effect of roadway surface upgrades. One study found the surface upgrades can reduce crashes by 35 percent on urban and rural roads.(155) Another study has found a 33-percent reduction in crashes on rural roads.(160)

Speed Reduction Effectiveness

No published speed evaluations for this treatment have been found.

Cost

Costs for roadway surface upgrades are extremely variable. The cost is based on depth of resurfacing, material, whether or not the old surface is to be milled, etc.

GATEWAY TREATMENT

Source: Iowa State University

Figure 183. Photo. Gateway treatment.

Description

Gateway treatments are a combination of signage and aesthetics that convey entrance to a community.(126,53)

Design Details

Common tools used in gateway treatments include welcome signs, textured pavements, and pavement markings. Most are placed on the roadside, but some can span over the roadway.(126,53)

Safety Effectiveness

Published CMFs indicate a 2-percent reduction in crashes (CMF = 0.98) resulting from gateway treatments.(161)

Speed Reduction Effectiveness

Gateway treatments have been shown to reduce 85th percentile speeds by 2 mph.(53)

Cost

The cost to construct a gateway treatment can vary greatly, depending on the design of the sign. For example, an intricate gateway treatment may cost almost $10,000, including labor and materials.(55)

APPENDIX C. SAFETY EFFECTS OF LANE-WIDTH AND SHOULDER-WIDTH COMBINATIONS ON RURAL, TWO-LANE ROADS (ADDITIONAL MODELING RESULTS)

Table 107. Negative binomial regression model estimation results for property damage only crashes (all severities).

Variable	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
ln_aadt	0.649	0.061	10.700	< 0.001	0.530	0.768
illinois	1.195	0.108	11.080	< 0.001	0.984	1.407
Lane10	1.047	0.227	4.610	< 0.001	0.602	1.493
Lane11	0.431	0.166	2.610	0.009	0.107	0.756
shoulder	-0.052	0.024	-2.140	0.032	-0.100	-0.004
ln10shld	-0.119	0.060	-1.980	0.048	-0.238	-0.001
ln11shld	-0.037	0.032	-1.160	0.246	-0.099	0.025
barrier	4.346	1.807	2.400	0.016	0.804	7.889
drvwy_den	0.016	0.007	2.340	0.019	0.003	0.029
solid_CL	0.563	0.215	2.620	0.009	0.143	0.984
dash1_CL	0.992	0.334	2.970	0.003	0.338	1.646
curve	0.253	0.076	3.340	0.001	0.105	0.402
_cons	-5.401	0.486	-11.120	< 0.001	-6.352	-4.449
ln(Length)	1	(exposure)		—	—	—
/lnalpha	-0.86727	0.117387	—	—	-1.09734	-0.63719
alpha	0.420098	0.049314	—	—	0.333757	0.528774

Log likelihood = -1473.751
Number of Observations = 877
LR chi2(12) = 425.44
Prob > chi2 = 0
Pseudo R2 = 0.1261
Likelihood-ratio test of alpha=0: chibar2(01) = 254.30 Prob>=chibar2 = 0.000
— Indicates Not Available

Table 108. Negative binomial regression model estimation results for single-vehicle crashes (all severities).

sv_all	Coefficient	Standard Error	z	P > z	95-percent Confidence Interval	
ln_aadt	0.514	0.059	8.67	< 0.001	0.398	0.631
illinois	1.128	0.103	10.9	< 0.001	0.925	1.331
Lane10	0.896	0.219	4.08	< 0.001	0.466	1.326
Lane11	0.413	0.160	2.58	0.010	0.099	0.727
shoulder	-0.064	0.024	-2.71	0.007	-0.111	-0.018
ln10shld	-0.090	0.058	-1.56	0.120	-0.202	0.023
ln11shld	-0.030	0.031	-0.99	0.323	-0.091	0.030
barrier	3.597	1.760	2.04	0.041	0.147	7.046
drvwy_den	0.017	0.007	2.630	0.009	0.004	0.030
solid_CL	0.478	0.208	2.300	0.022	0.070	0.886
dash1_CL	0.923	0.324	2.850	0.004	0.288	1.559
curve	0.246	0.074	3.330	0.001	0.101	0.391
_cons	-4.174	0.469	-8.890	< 0.001	-5.094	-3.254
ln(Length)	1	(exposure)		—	—	—
/lnalpha	-0.94318	0.120937	—	—	-1.180212	-0.7061473
alpha	0.389388	0.047092	—	—	0.3072135	0.493542

Log likelihood = -1493.2046
Number of Observations = 877
LR chi2(12) = 408.55
Prob > chi2 = 0
Pseudo R2 = 0.1203
Likelihood-ratio test of alpha=0: chibar2(01) = 229.65 Prob>=chibar2 = 0.000
— Indicates Not Available

Table 109. Negative binomial regression model estimation results for multiple-vehicle crashes (all severities).

mv_all	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
ln_aadt	1.482	0.110	13.430	< 0.001	1.266	1.698
illinois	0.628	0.171	3.680	< 0.001	0.294	0.962
Lane10	0.769	0.470	1.640	0.102	-0.153	1.690
Lane11	0.199	0.306	0.650	0.515	-0.400	0.798
shoulder	0.007	0.040	0.160	0.872	-0.073	0.086
ln10shld	-0.223	0.132	-1.690	0.092	-0.481	0.036
ln11shld	-0.006	0.053	-0.110	0.910	-0.110	0.098
barrier	5.690	2.653	2.150	0.032	0.491	10.889
drvwy_den	-0.002	0.012	-0.160	0.875	-0.024	0.021
solid_CL	0.422	0.343	1.230	0.218	-0.250	1.094
dash1_CL	0.641	0.542	1.180	0.237	-0.421	1.703
curve	-0.013	0.126	-0.100	0.921	-0.259	0.234
_cons	-13.401	0.909	-14.750	< 0.001	-15.182	-11.620
ln(Length)	1	(exposure)		—	—	—
/lnalpha	-1.1353	0.372991	—	—	-1.866348	-0.4042488
alpha	0.321326	0.119852	—	—	0.1546876	0.667478

Log likelihood = -649.6117
Number of Observations = 877
LR chi2(12) = 213.52
Prob > chi2 = 0
Pseudo R2 = 0.1411
Likelihood-ratio test of alpha=0: chibar2(01) = 12.89 Prob>=chibar2 = 0.000
— Indicates Not Available

Table 110. Negative binomial regression model estimation results for key lane width/shoulder width crashes (all severities).

key_kabco	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
ln_aadt	0.606	0.059	10.27	< 0.001	0.490	0.721
illinois	1.083	0.099	10.98	< 0.001	0.890	1.277
Lane10	0.903	0.223	4.04	< 0.001	0.465	1.341
Lane11	0.519	0.161	3.23	0.001	0.205	0.834
shoulder	-0.052	0.023	-2.26	0.024	-0.097	-0.007
ln10shld	-0.076	0.056	-1.37	0.172	-0.185	0.033
ln11shld	-0.060	0.030	-1.97	0.049	-0.119	0.000
barrier	4.305	1.888	2.28	0.023	0.604	8.006
drvwy_den	0.012	0.007	1.88	0.060	-0.001	0.025
solid_CL	0.471	0.209	2.26	0.024	0.062	0.880
dash1_CL	0.800	0.323	2.47	0.013	0.166	1.433
curve	0.231	0.074	3.14	0.002	0.087	0.376
_cons	-4.613	0.466	-9.9	<0.001	-5.527	-3.700
ln(Length)	1	(exposure)		—	—	—
/lnalpha	-0.72081	0.102919	—	—	-0.92253	-0.5191
alpha	0.486356	0.050055	—	—	0.397512	0.595057

Log likelihood = -1684.8577
Number of Observations = 877
LR chi2(12) = 395.990
Prob > chi2 = 0.000
Pseudo R2 = 0.105
Likelihood-ratio test of alpha=0: chibar2(01) = 395.21 Prob>=chibar2 = 0.000
— Indicates Not Available

Table 111. Negative binomial regression model estimation results for single-vehicle crashes (fatal-plus-injury).

sv_fi	Coefficient	Standard. Error	z	P > z	95-Percent Confidence Interval	
ln_aadt	0.562	0.112	5.01	< 0.001	0.342	0.783
illinois	0.427	0.189	2.26	0.024	0.057	0.798
Lane10	-0.035	0.428	-0.08	0.935	-0.874	0.804
Lane11	0.193	0.305	0.63	0.528	-0.406	0.791
shoulder	-0.116	0.045	-2.58	0.010	-0.203	-0.028
ln10shld	0.063	0.106	0.6	0.551	-0.144	0.271
ln11shld	-0.023	0.061	-0.39	0.699	-0.142	0.095
barrier	0.576	3.244	0.18	0.859	-5.783	6.934
drvwy_den	0.007	0.013	0.590	0.553	-0.017	0.032
solid_CL	0.530	0.369	1.440	0.151	-0.194	1.253
dash1_CL	0.345	0.615	0.560	0.575	-0.860	1.550
curve	0.081	0.141	0.580	0.563	-0.194	0.357
_cons	-5.385	0.882	-6.100	<0.001	-7.114	-3.656
ln(Length)	1	(exposure)		—	—	—
/lnalpha	-1.10368	0.491638	—	—	-2.06727	-0.1400852
alpha	0.331649	0.163051	—	—	0.1265307	0.8692842

Log likelihood = -579.36383
Number of Observations = 877
LR chi2(12) = 64.28
Prob > chi2 = 0
Pseudo R2 = 0.0526
Likelihood-ratio test of alpha=0: chibar2(01) = 6.35 Prob>=chibar2 = 0.006
— Indicates Not Available

Table 112. Negative binomial regression model estimation results for multiple-vehicle crashes (fatal-plus-injury).

mv_fi	Coefficient	Standard Error	z	P > z	95-percent Confidence Interval	
ln_aadt	1.142	0.162	7.060	< 0.001	0.825	1.459
illinois	0.557	0.248	2.240	0.025	0.071	1.043
Lane10	0.427	0.992	0.430	0.667	-1.517	2.371
Lane11	0.762	0.481	1.580	0.114	-0.182	1.705
shoulder	0.146	0.059	2.470	0.013	0.030	0.262
ln10shld	-0.185	0.219	-0.840	0.398	-0.613	0.244
ln11shld	-0.100	0.078	-1.290	0.196	-0.252	0.052
barrier	5.476	4.069	1.350	0.178	-2.498	13.450
drvwy_den	-0.011	0.019	-0.570	0.570	-0.048	0.026
solid_CL	0.129	0.559	0.230	0.817	-0.966	1.225
dash1_CL	0.533	0.836	0.640	0.524	-1.106	2.173
curve	0.126	0.186	0.680	0.498	-0.238	0.490
_cons	-12.279	1.343	-9.140	<0.001	-14.911	-9.646
ln(Length)	1	(exposure)		—	—	—
/lnalpha	-1.18079	0.886305	—	—	-2.917915	0.5563366
alpha	0.307036	0.272128	—	—	0.0540463	1.744271

Log likelihood = -374.64352
Number of Observations = 877
LR chi2(12) = 79.46
Prob > chi2 = 0
Pseudo R2 = 0.0959
Likelihood-ratio test of alpha=0: chibar2(01) = 1.73 Prob>=chibar2 = 0.094
— Indicates Not Available

Table 113. Negative binomial regression model estimation results for key lane-width/shoulder-width crashes (fatal-plus-injury).

key_kabc	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
ln_aadt	0.598	0.114	5.27	< 0.001	0.376	0.821
illinois	0.523	0.180	2.9	0.004	0.170	0.875
Lane10	0.080	0.458	0.17	0.862	-0.819	0.978
Lane11	0.552	0.319	1.73	0.084	-0.073	1.178
shoulder	-0.070	0.044	-1.6	0.109	-0.155	0.015
ln10shld	0.068	0.105	0.65	0.518	-0.138	0.274
ln11shld	-0.093	0.060	-1.55	0.121	-0.211	0.025
barrier	0.314	3.738	0.08	0.933	-7.014	7.641
drvwy_den	0.009	0.013	0.67	0.501	-0.017	0.034
solid_CL	0.288	0.397	0.72	0.469	-0.490	1.066
dash1_CL	0.416	0.629	0.66	0.508	-0.815	1.648
curve	0.034	0.147	0.23	0.818	-0.255	0.322
_cons	-5.392	0.888	-6.07	<0.001	-7.132	-3.651
ln(Length)	1	(exposure)		—	—	—
/lnalpha	0.481668	0.153744		—	0.180335	0.783001
alpha	1.618773	0.248877	—	—	1.197619	2.18803

Log likelihood = -817.20472
Number of Observations = 877
LR chi2(12) = 66
Prob > chi2 = 0.0000
Pseudo R2 = 0.0388
Likelihood-ratio test of alpha=0: chibar2(01) = 144.74 Prob>=chibar2 = 0.000
— Indicates Not Available

Table 114. Negative binomial regression model estimation results for single-vehicle crashes (PDO).

sv_o	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
ln_aadt	0.515	0.065	7.9	< 0.001	0.387	0.643
illinois	1.307	0.117	11.17	< 0.001	1.077	1.536
Lane10	1.029	0.240	4.3	< 0.001	0.560	1.499
Lane11	0.427	0.175	2.44	0.015	0.083	0.770
shoulder	-0.054	0.026	-2.04	0.041	-0.105	-0.002
ln10shld	-0.113	0.064	-1.77	0.077	-0.238	0.012
ln11shld	-0.031	0.034	-0.9	0.370	-0.097	0.036
barrier	4.386	1.928	2.27	0.023	0.606	8.166
drvwy_den	0.018	0.007	2.490	0.013	0.004	0.032
solid_CL	0.499	0.230	2.170	0.030	0.048	0.951
dash1_CL	1.031	0.356	2.900	0.004	0.333	1.728
curve	0.288	0.081	3.570	<0.001	0.130	0.447
_cons	-4.589	0.519	-8.840	<0.001	-5.606	-3.572
ln(Length)	1	(exposure)		—	—	—
/lnalpha	-0.76859	0.121186	—	—	-1.006113	-0.5310743
alpha	0.463665	0.05619	—	—	0.3656375	0.587973

Log likelihood = -1390.4041
Number of Observations = 877
LR chi2(12) = 397.58
Prob > chi2 = 0
Pseudo R2 = 0.1251
Likelihood-ratio test of alpha=0: chibar2(01) = 239.28 Prob>=chibar2 = 0.000
— Indicates Not Available

Table 115. Negative binomial regression model estimation results for multiple-vehicle crashes (PDO).

mv_o	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
ln_aadt	1.727	0.144	12.010	< 0.001	1.445	2.009
illinois	0.671	0.222	3.020	0.003	0.236	1.105
Lane10	0.686	0.542	1.270	0.206	-0.376	1.749
Lane11	-0.198	0.383	-0.520	0.605	-0.948	0.552
shoulder	-0.095	0.052	-1.810	0.071	-0.197	0.008
ln10shld	-0.186	0.167	-1.120	0.264	-0.514	0.141
ln11shld	0.068	0.070	0.980	0.329	-0.069	0.205
barrier	5.742	3.298	1.740	0.082	-0.722	12.207
drvwy_den	0.004	0.014	0.290	0.775	-0.024	0.032
solid_CL	0.568	0.420	1.350	0.176	-0.254	1.391
dash1_CL	0.653	0.682	0.960	0.338	-0.684	1.991
curve	-0.096	0.161	-0.590	0.552	-0.412	0.220
_cons	-15.436	1.179	-13.090	<0.001	-17.747	-13.124
ln(Length)	1	(exposure)		—	—	—
/lnalpha	-0.79447	0.433001	—	—	-1.643133	0.0542011
alpha	0.451823	0.19564	—	—	0.1933733	1.055697

Log likelihood = -469.42118
Number of Observations = 877
LR chi2(12) = 178.04
Prob > chi2 = 0
Pseudo R2 = 0.1594
Likelihood-ratio test of alpha=0: chibar2(01) = 9.54 Prob>=chibar2 = 0.001
— Indicates Not Available

Table 116. Negative binomial regression model estimation results for key lane-width/shoulder-width crashes (PDO).

key_o	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
ln_aadt	0.615	0.066	9.37	< 0.001	0.486	0.743
illinois	1.276	0.113	11.25	< 0.001	1.054	1.498
Lane10	1.059	0.245	4.33	< 0.001	0.579	1.539
Lane11	0.494	0.177	2.8	0.005	0.148	0.841
shoulder	-0.046	0.026	-1.8	0.072	-0.097	0.004
ln10shld	-0.104	0.063	-1.67	0.096	-0.227	0.018
ln11shld	-0.049	0.034	-1.45	0.147	-0.115	0.017
barrier	5.253	2.057	2.55	0.011	1.221	9.286
drvwy_den	0.013	0.007	1.85	0.065	-0.001	0.028
solid_CL	0.544	0.232	2.35	0.019	0.090	0.999
dash1_CL	0.913	0.358	2.55	0.011	0.211	1.614
curve	0.292	0.081	3.6	< 0.001	0.133	0.451
_cons	-5.167	0.523	-9.88	< 0.001	-6.192	-4.141
ln(Length)	1	(exposure)		—	—	—
/lnalpha	-0.5778	0.107196	—	—	-0.7879	-0.3677
alpha	0.561129	0.060151	—	—	0.454797	0.692322

Log likelihood = -1524.6018
Number of Observations = 877
LR chi2(12) = 391.03
Prob > chi2 = 0
Pseudo R2 = 0.1137
Likelihood-ratio test of alpha=0: chibar2(01) = 363.82 Prob>=chibar2 = 0.000
— Indicates Not Available

Table 117. Multinomial logit model estimation results for total crashes (all types and severities)—base outcome: PDO.

Severity	Variable	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
0	(base outcome)	—	—	—	—	—	—
1	ln_aadt	0.103	0.190	0.540	0.588	-0.270	0.476
	illinois	-2.615	0.301	-8.700	0.000	-3.205	-2.026
	Lane10	-0.993	0.914	-1.090	0.277	-2.784	0.797
	Lane11	-0.125	0.546	-0.230	0.819	-1.195	0.945
	shoulder	-0.079	0.071	-1.110	0.267	-0.218	0.060
	ln10shld	0.216	0.173	1.250	0.211	-0.123	0.555
	ln11shld	0.023	0.088	0.260	0.797	-0.150	0.195
	barrier	-5.154	7.644	-0.670	0.500	-20.137	9.829
	drvway_den	-0.008	0.025	-0.330	0.739	-0.058	0.041
	solid_CL	-0.905	0.767	-1.180	0.238	-2.409	0.599
	dash1_CL	0.857	1.031	0.830	0.406	-1.163	2.877
	curve	-0.377	0.237	-1.590	0.111	-0.842	0.087
	_cons	-1.450	1.448	-1.000	0.316	-4.288	1.387
2	ln_aadt	0.038	0.134	0.280	0.780	-0.226	0.301
	illinois	-0.007	0.257	-0.030	0.980	-0.511	0.498
	Lane10	-1.107	0.630	-1.760	0.079	-2.341	0.127
	Lane11	-0.172	0.368	-0.470	0.641	-0.893	0.550
	shoulder	0.082	0.053	1.540	0.124	-0.023	0.186
	ln10shld	0.133	0.152	0.880	0.380	-0.164	0.431
	ln11shld	-0.032	0.070	-0.450	0.650	-0.170	0.106
	barrier	1.097	4.403	0.250	0.803	-7.532	9.726
	drvway_den	-0.022	0.017	-1.300	0.192	-0.054	0.011
	solid_CL	-0.331	0.495	-0.670	0.504	-1.301	0.639
	dash1_CL	-0.595	0.771	-0.770	0.441	-2.107	0.917
	curve	-0.218	0.168	-1.300	0.194	-0.547	0.111
	_cons	-2.529	1.099	-2.300	0.021	-4.683	-0.375
3	ln_aadt	0.257	0.168	1.530	0.126	-0.073	0.587
	illinois	0.613	0.393	1.560	0.119	-0.157	1.383
	Lane10	-0.057	0.693	-0.080	0.934	-1.416	1.302
	Lane11	0.443	0.481	0.920	0.357	-0.500	1.386
	shoulder	0.085	0.073	1.160	0.248	-0.059	0.229
	ln10shld	-0.069	0.215	-0.320	0.747	-0.491	0.352
	ln11shld	-0.097	0.096	-1.010	0.312	-0.284	0.091
	barrier	-1.734	5.888	-0.290	0.768	-13.273	9.806
	drvway_den	-0.022	0.019	-1.130	0.258	-0.060	0.016
	solid_CL	0.884	0.485	1.820	0.068	-0.067	1.836
	dash1_CL	-1.950	1.012	-1.930	0.054	-3.934	0.034
	curve	0.179	0.205	0.870	0.382	-0.222	0.580
	_cons	-5.679	1.441	-3.940	0.000	-8.503	-2.855

Severity	Variable	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
4	ln_aadt	0.228	0.351	0.650	0.516	-0.460	0.917
	illinois	-0.130	0.700	-0.190	0.853	-1.501	1.242
	Lane10	1.177	1.687	0.700	0.485	-2.129	4.484
	Lane11	1.441	1.082	1.330	0.183	-0.681	3.562
	shoulder	0.207	0.156	1.320	0.186	-0.099	0.513
	ln10shld	-0.671	0.962	-0.700	0.486	-2.557	1.216
	ln11shld	-0.211	0.189	-1.110	0.265	-0.581	0.160
	barrier	-3.203	12.442	-0.260	0.797	-27.589	21.183
	drvway_den	-0.054	0.050	-1.090	0.276	-0.152	0.043
	solid_CL	0.672	1.006	0.670	0.504	-1.300	2.644
	dash1_CL	0.528	1.968	0.270	0.788	-3.330	4.386
	curve	0.403	0.425	0.950	0.343	-0.430	1.236
	_cons	-7.442	3.027	-2.460	0.014	-13.376	-1.508

Log likelihood = -1543.1747
Number of observation = 2397
LR chi2(48) = 198.440
Prob > chi2 = 0.000
Pseudo R2 = 0.060
— Indicates Not Available

Table 118. Multinomial logit model estimation results for fatal-plus-injury crashes (all types)—base outcome: possible injury.

severity_rev	Variable	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
1	(base outcome)	—	—	—	—	—	—
2	ln_aadt	-0.273	0.231	-1.180	0.237	-0.726	0.180
	illinois	2.531	0.388	6.530	<0.001	1.771	3.290
	Lane10	0.011	1.163	0.010	0.992	-2.267	2.290
	Lane11	0.044	0.689	0.060	0.949	-1.305	1.394
	shoulder	0.163	0.091	1.780	0.075	-0.016	0.342
	ln10shld	-0.191	0.250	-0.770	0.444	-0.682	0.299
	ln11shld	-0.063	0.118	-0.530	0.594	-0.294	0.168
	barrier	10.549	9.177	1.150	0.250	-7.438	28.537
	drvway_den	-0.006	0.031	-0.200	0.840	-0.067	0.054
	solid_CL	0.618	0.866	0.710	0.475	-1.079	2.316
	dash1_CL	-1.659	1.326	-1.250	0.211	-4.258	0.940
	curve	0.147	0.304	0.490	0.627	-0.448	0.742
	_cons	0.604	1.785	0.340	0.735	-2.895	4.103
3	ln_aadt	-0.057	0.263	-0.220	0.829	-0.572	0.459
	illinois	3.230	0.505	6.400	<0.001	2.240	4.220
	Lane10	1.030	1.226	0.840	0.401	-1.372	3.432
	Lane11	0.758	0.781	0.970	0.332	-0.773	2.289
	shoulder	0.184	0.111	1.660	0.096	-0.033	0.401
	ln10shld	-0.389	0.299	-1.300	0.192	-0.974	0.196
	ln11shld	-0.155	0.141	-1.100	0.273	-0.432	0.122
	barrier	7.135	10.398	0.690	0.493	-13.245	27.514
	drvway_den	-0.005	0.033	-0.150	0.884	-0.069	0.059
	solid_CL	1.739	0.882	1.970	0.049	0.010	3.468
	dash1_CL	-3.028	1.529	-1.980	0.048	-6.025	-0.030
	curve	0.542	0.336	1.610	0.107	-0.116	1.199
	_cons	-2.654	2.127	-1.250	0.212	-6.823	1.515

severity_rev	Variable	Coefficient	Standard Error	z	P > z	95-Percent Confidence Interval	
4	ln_aadt	-0.047	0.406	-0.120	0.907	-0.843	0.748
	illinois	2.492	0.779	3.200	0.001	0.966	4.017
	Lane10	2.603	2.183	1.190	0.233	-1.676	6.882
	Lane11	1.840	1.247	1.480	0.140	-0.604	4.284
	shoulder	0.310	0.178	1.740	0.082	-0.039	0.660
	ln10shld	-1.145	1.307	-0.880	0.381	-3.707	1.417
	ln11shld	-0.289	0.217	-1.330	0.183	-0.715	0.137
	barrier	4.281	15.898	0.270	0.788	-26.879	35.442
	drvway_den	-0.041	0.057	-0.720	0.472	-0.152	0.070
	solid_CL	1.370	1.227	1.120	0.264	-1.035	3.775
	dash1_CL	-0.305	2.244	-0.140	0.892	-4.702	4.092
	curve	0.757	0.496	1.530	0.127	-0.216	1.730
	_cons	-4.744	3.408	-1.390	0.164	-11.423	1.936

Log likelihood = -464.69741
Number of Observations = 426
LR chi2(36) = 112.18
Prob > chi2 = 0
Pseudo R2 = 0.1077
— Indicates Not Available

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