Self-Enforcing Roadways:
A Guidance Report

U.S. Department of Transportation
Federal Highway Administration

FOREWORD

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 collector roads have the highest speeding-related fatality rate on the basis of miles driven per vehicle. A self-enforcing road (sometimes referred to as a "self-explaining roadway") is a roadway that is planned and designed to encourage drivers to select operating speeds in harmony with the posted speed limit. Properly designed self-enforcing roadways can be effective in producing speed compliance and may contribute to less severe crash outcomes.

The purpose of this report is to provide guidance on how to produce self-enforcing roadways. The concepts can be applied to planned and existing roadways. This report should be useful to transportation professionals, State departments of transportation, and researchers interested in designing and/or retrofitting roadways to induce drivers to drive at more appropriate speeds.

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TECHNICAL REPORT DOCUMENTATION PAGE

1. Report No. FHWA-HRT-17-098	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle Self-Enforcing Roadways: A Guidance Report		5. Report Date January 2018	
		6. Performing Organization Code	
7. Author(s) Eric Donnell, Kristin Kersavage, and Lisa Fontana Tierney		8. Performing Organization Report No.	
9. Performing Organization Name and Address Institute of Transportation Engineers 1627 Eye Street NW, Suite 600 Washington, DC 20006 Penn State University 212 Sackett Building University Park, PA 16802		10. Work Unit No.	
		11. Contract or Grant No. DTFH61-13-D-00026/0007	
12. Sponsoring Agency Name and Address U.S. Department of Transportation Federal Highway Administration Turner-Fairbank Highway Research Center 6300 Georgetown Pike McLean, VA 22101		13. Type of Report and Period Covered Final Report; July 2015–November 2017	
		14. Sponsoring Agency Code:	
15. Supplementary Notes The Task Order Manager for this report is Abdul Zineddin (HRDS-10).			
16. Abstract The objective of this project was to develop a guidance report to identify methods that may produce self-enforcing, or self-explaining, roadways during the geometric design process. While safety performance associated with these methods is not well understood yet, an implied outcome of effective speed management is that less severe crashes will result via the application of self-enforcing, or self-explaining, road design principles. Six self-enforcing road concepts and the processes needed to implement the concepts when designing or evaluating existing two-lane rural highways are identified and described in this document. It is anticipated that the concepts may be used to design roadways that produce operating speeds consistent with the desired operating speeds of the roadway. The six methods include: (1) the speed feedback loop process, (2) the inferred design speed approach, (3) design consistency methods, (4) applying geometric design criteria, (5) using a combination of signs and pavement markings, and (6) setting rational speed limits.			
17. Key Words Self-enforcing road, self-explaining road, two-lane rural highway, operating speed, speed limit, design speed, speed management, safety		18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161. http://www.ntis.gov	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages: 118	22. Price

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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/m2	cd/m2
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/m2	candela/m2	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

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

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LIST OF ABBREVIATIONS

AADT	annual average daily traffic (vehicles per day)
AASHTO	American Association of State Highway and Transportation Officials
DCM	Design Consistency Module
EB	empirical Bayes
FHWA	Federal Highway Administration
HSM	*Highway Safety Manual*
HSO	horizontal sight line offset
IHSDM	Interactive Highway Safety Design Model
ITE	Institute of Transportation Engineers
MUTCD	*Manual on Uniform Traffic Control Devices*
NHTSA	National Highway Traffic Safety Administration
OLS	ordinary least squares
SHRP2	Second Strategic Highway Research Program
SSD	stopping sight distance
TAC	Transportation Association of Canada
TRB	Transportation Research Board

EXECUTIVE SUMMARY

Reducing the number of fatal and injury crashes in the United States is a high priority among Federal, State, and local transportation agencies. Crashes occurring in rural areas comprise most of the total crashes, many of which can be attributed to speeding. Due to the substantial number of speeding-related crashes in rural areas, traffic safety improvement programs should focus on crash-reduction strategies in these rural areas, with the most effective speed-management programs focusing on reducing speeding-related crashes on moderate- and high-speed, two-lane rural highways. Therefore, this guidance document focuses on methods to mitigate speeding-related crashes on two-lane rural highways with posted speed limits of 35 mph (56.4 km/h) or greater.

Because speeding is a complex problem that involves the interaction of many factors, successfully mitigating speeding-related crashes requires integration and coordination among engineering, enforcement, and education. From an engineering perspective, a design concept referred to as "self-enforcing roadways" has been developed to guide appropriate road-user behavior. A self-enforcing road, also called a "self-explaining roadway," is a roadway that is planned and designed to encourage drivers to select operating speeds consistent with the posted speed limit. To affect speed compliance, designers typically use geometric elements that encourage drivers to select operating speeds that are appropriate for the intended purpose of the roadway. The ideal is for operating speeds and posted speed limits to be in harmony with the roadway's geometric design speed.

This guidance report identifies methods that may produce self-enforcing, or self-explaining, roadways during the geometric design process. While safety performance associated with these methods is not yet well understood, an implied outcome of effective speed management is that less severe crashes will result via the application of self-enforcing, or self-explaining, road-design principles. This report identifies and describes six self-enforcing road concepts and the processes needed to implement these concepts when designing or evaluating existing two-lane rural highways. It is anticipated that the concepts may be used to design roadways that produce operating speeds consistent with the desired operating speeds of the roadway. The six concepts, further described below, include (1) the speed feedback loop process, (2) the inferred design speed approach, (3) design consistency methods, (4) the application of existing geometric design criteria, (5) the combination of signs and pavement markings, and (6) the setting of rational speed limits:

1. **The speed feedback loop process** requires checking for consistency among geometric design elements and anticipated operating speeds to determine an appropriate posted speed limit. Use of a speed feedback loop during the design process involves speed prediction during preliminary engineering and compares the expected operating speed to the geometric design speed. A desirable outcome occurs when the expected operating speed is equal to, or nominally lower than, the designated design speed.

2. **The inferred design speed approach** evaluates the geometric design of a roadway by determining the maximum speed for which all design speed-related criteria are met. The

inferred design speed is then compared to the anticipated or measured operating speeds and the posted speed limit using graphical methods.

3. **Design consistency methods**, such as the application of operating speed prediction models, may be used to establish posted speed limits based on predicted operating speed. Operating speed models aid in determining the effects that geometric design features have on driver speed choice along a roadway. Design consistency methods can be applied either manually, using a series of equations, or using a computer model, such as the Design Consistency Module of the Federal Highway Administration's (FHWA) Interactive Highway Safety Design Model. (FHWA 2016a)

4. **The application of existing geometric design criteria** includes the use of features with a known relationship to operating speed. Understanding how the design values affect speed, including the spatial relationships between different geometric elements, may affect driver speed choice along an alignment and promote operations that are consistent with the posted speed limit.

5. **The combination of signs and pavement markings** can be used to manage speeds on existing roadways when major reconstruction of the road is not feasible. Traffic control devices, such as signs and pavement markings, communicate information to drivers regarding speed choice and may encourage drivers to select speeds that are more in harmony with the posted speed limit on the roadway. Many signs and pavement markings have been evaluated and are associated with driver speed choice. Several examples of these signs and pavement markings that are known to reduce operating speeds on two-lane rural highways are described in this report.

6. **The setting of rational speed limits** consists of choosing a posted speed limit that is reasonable, rational, and consistent with the features of the roadway. A Web-based tool that can be used to provide guidance regarding appropriate posted speed limits is the FHWA's USLIMITS2. (FHWA 2016b) The program determines rational speed limits using an expert system, which includes a series of decision rules and procedures that are applied based on user input.

These methods can be applied individually or in combination for planned and existing two-lane rural highways. For new or major reconstruction activities, the speed feedback loop process, the inferred design speed approach, the design consistency method, and the application of existing geometric design criteria should be considered the most effective self-enforcing, or self-explaining, design concepts. These methods should be incorporated early in the planning and design process to ensure that roads are designed to produce operating speeds similar to the intended operating speeds on the roadway. However, when major reconstruction activities cannot be—or are not—planned to occur on a roadway, existing roadways can be retrofitted with a combination of signs and pavement markings to produce a self-enforcing, or self-explaining, roadway. The setting of rational speed limits may be considered for both planned and existing roadways.

CHAPTER 1. INTRODUCTION

Reducing the number of fatal and injury crashes in the United States is a high priority among Federal, State, and local transportation agencies. The number of fatal crashes has declined from 36,254 (40,716 fatalities) in 1994 to 29,989 (32,675 fatalities) in 2014. (National Highway Traffic Safety Administration (NHTSA) 2015) Based on 2013 crash statistics, approximately 53 percent of the fatal crashes occurred in rural areas. The fatal crash rate in rural areas is 1.88 per 100 million vehicle-miles traveled, which is more than 2.5 times the urban fatal crash rate of 0.73. This indicates that traffic safety improvement programs with strategies for rural areas may be especially successful in reducing crash frequency and severity.

Of particular interest among the total traffic-related fatalities in the United States are those attributed to speeding, which comprised 29 percent (9,613) of the total fatalities. Speeding-related crashes are those defined as driving too fast for conditions or exceeding the posted speed limit. (NHTSA 2015a) Among the rural traffic fatalities, 30 percent (5,346 of 17,696) were codified as speeding-related. In 2014, the percentage of speeding-related crashes that occurred on roadways with a speed limit of 25 mph (40.3 km/h) or less was 7 percent; the percentage of speeding-related crashes on roadways with a speed limit higher than 25 mph (40.3 km/h), but less than or equal to 40 mph (64.4 km/h), was 28 percent; the percentage of speeding-related crashes that occurred on roadways with a speed limit between 45 (72.5 km/h) and 80 mph (128.8 km/h) was 65 percent. (NHTSA 2015) Collectively, these data suggest there is value in effective speed-management programs to reduce speeding-related crashes on moderate- and high-speed, two-lane rural highways.

Speeding is a complex issue that involves the interaction of many factors, including public attitudes, road-user behavior, vehicle performance, roadway design characteristics, posted speed limits, enforcement strategies, and judicial decisions. To be successful in mitigating speeding-related crashes, engineering, enforcement, and education must be integrated and coordinated.

From an engineering perspective, one design concept that has been developed to successfully guide appropriate road-user behavior is known as "self-enforcing roadways." A self-enforcing road, also called a "self-explaining roadway," is a roadway that is planned and designed to encourage drivers to select operating speeds consistent with the posted speed limit. Road designers use geometric elements to accomplish their goal of encouraging a target operating speed. Properly designed self-enforcing roadways can be effective in producing speed compliance and may contribute to less severe crash outcomes. This report provides guidance on how to produce self-enforcing roadways; the concepts can be applied to both planned and existing roadways.

Two-lane rural highways with posted speed limits of 35 mph (56.4 km/h) or greater are the focus of this report. A significant body of published literature related to operating speeds and safety is available for high-speed roads, which are defined as roadways with design speeds that are 50 mph (80.5 km/h) or greater. A report by Liu and Chen (2009) found that speeding-related crashes are more often coded as exceeding the posted speed limit on roadways with posted speed limits of 50 mph (80.5 km/h) or less, which indicates that the guidance included in this report should cover moderate-speed roadways with posted speed limits in the range of 35 to 50 mph (56.4 to

80.5 km/h). For lower-speed roadways (posted speed limits less than 35 mph (56.4 km/h)), information from the Institute of Transportation Engineers' (ITE) *Traffic Calming ePrimer* can be used to provide speed-management solutions. (ITE 2017)

The remainder of this report is organized as follows:

- Chapter 2 examines the relationship between speed and safety, describing the interaction of crash frequency and severity with speed.

- Chapter 3 discusses the relationship between speed and geometric design, including horizontal and vertical alignments and cross-section elements. It contains the objective information used to inform many of the concepts described in chapter 5.

- Chapter 4 describes six self-enforcing road concepts and the processes needed to implement the concepts when designing or assessing two-lane rural highways.

- Chapter 5 presents two case study examples that apply several of the self-enforcing roadway concepts.

- Chapter 6 discusses implementation of the self-enforcing road concepts.

Readers interested in applying self-enforcing, or self-explaining, roadway design methods in practice are recommended to focus on chapters 4 and 5. Those interested in learning additional technical details underlying the self-enforcing roadway methods described in this report are recommended to read chapters 3 through 5.

CHAPTER 2. RELATIONSHIP BETWEEN SPEED AND SAFETY

Speed–safety relationships are often described with emphasis on crash frequency, severity outcomes, and various speed metrics. This section of the report briefly describes the relationships between crash severity and operating speed, crash frequency and operating speed, and speed variance and safety performance. The purpose of this section is to describe how the safety performance of a roadway may change as a function of operating speeds.

CRASH SEVERITY–SPEED RELATIONSHIPS

With regard to crash severity, the operating speed of motor vehicles directly affects the crash outcome. In this case, higher vehicle operating speeds are associated with more severe crashes. This is the result of the kinetic energy dissipated during a crash event. (SWOV 2009) The equation in figure 1 shows the relationship between kinetic energy and speed:

$$Kinetic\ Energy = \frac{1}{2}mv^2$$

Figure 1. Equation. Relationship between kinetic energy, mass, and speed that occurs during a crash.

Where:
  m = mass of object (lbs (kg)).
  v = speed of object (ft/s (m/s)).

The kinetic energy is associated with the square of the operating speed; thus, higher operating speeds result in more severe crash outcomes. An example of this relationship was demonstrated by Rosén et al. (2011) in a study of pedestrian fatality risk when struck by a motor vehicle. The authors showed that the fatality risk of pedestrians increases as the impact speed increases. For example, when a vehicle is traveling 24.85 mph (40 km/h), the fatality risk of a pedestrian is approximately 10 percent. However, when a vehicle is operating at 62.14 mph (100 km/h) at the time of impact, the pedestrian's fatality risk is approximately 100 percent. Therefore, pedestrians have a greater probability of survival when the impact speed with a passenger car is lower.

The equation in figure 2, developed by Nilsson in 1982, shows the relationship between the expected change in the number of injury crashes as a function of average speed change on roads based on kinetic laws. (SWOV 2009, Elvik 2009)

$$LO_2 = LO_1 \left(\frac{v_2}{v_1}\right)^2$$

Figure 2. Equation. Expected number of injury crashes because of a change in the average operating speed. (Nilsson 1982)

Where:
    LO_2 = number of injury crashes after speed change.
    LO_1 = initial number of injury crashes (before speed change).
    v_2 = average speed in the after period (mph or km/h).
    v_1 = average speed in the before period (mph or km/h).

An increase in the average speed in the after-period is associated with an increase in the expected number of injury crashes after the average speed increase. However, if the after-period average speed decreases, the expected number of injury crashes after the speed change decreases. The exponent in figure 2 can be modified based on the severity outcome considered. The power of 2 shown in figure 2 relates to the number of injury crashes. Changing the power of 2 to a power of 3 can be used to predict the number of severe injury crashes; changing the power to a value of 4 is associated with fatal crashes. (SWOV 2009)

A more recent study by Elvik in 2009, using figure 2 as the basis, produced new exponents that differentiated rural and urban road types when estimating how injury crashes may change as a function of average speed changes. Table 1 lists the various crash severity types and the associated exponents for the rural roads. (SWOV 2009, Elvik 2009)

Table 1. Power function exponents for various crash severities for rural roads/freeways. (Elvik 2009, table 18)

Crash Severity	Best Estimate Exponent	95% Confidence Interval
Fatal accidents	4.1	(2.9, 5.3)
Fatalities	4.6	(4.0, 5.2)
Serious-injury accidents	2.6	(–2.7, 7.9)
Seriously injured road users	3.5	(0.5, 5.5)
Slight-injury accidents	1.1	(0.0, 2.2)
Slightly injured road users	1.4	(0.5, 2.3)
Injury accidents—all	1.6	(0.9, 2.3)
Injured road users—all	2.2	(1.8, 2.6)
Property damage only accidents	1.5	(0.1, 2.9)

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The exponents shown in table 1 can be substituted into figure 3 to predict the number of each crash type after a speed change.

$$LO_2 = LO_1 \left(\frac{v_2}{v_1}\right)^n$$

Figure 3. Equation. Expected number of varying crash types as a result of a change in average operating speed.

Where *n* is the best estimated exponent from table 1.

Similar findings were reported by Kockelman et al. (2006). The study determined that speed limit increases were associated with more severe injuries in a crash. (Kockelman et al. 2006)

Table 2 illustrates the change in probability of fatal injuries and the change in fatal injury count as a result of an increase in the posted speed limit.

Table 2. Effects of speed increases on severity of crashes. (Kockelman et al. 2006)

Increase in Speed Limit (mph (km/h))	Change in Probability of Fatal Injury (%)	Total Change in Fatal Injury Count (%)
55 (88.6) to 65 (104.7)	+24	+28
65 (104.7) to 75 (120.8)	+12	+13

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A study by Malyshkina and Mannering (2008) determined the effects of posted speed limit increases on crash severity outcomes, including crashes that occurred on rural country roads, rural State routes, rural city streets, and rural U.S. routes. In general, the authors found that increases in the posted speed limit significantly increased the likelihood a crash would result in an injury or fatality for rural roads. (Malyshkina and Mannering 2008) Table 3 shows the probability of fatal and injury crashes associated with an increase in the posted speed limit.

As shown in table 3, the probability of fatal and injury crashes increases as the posted speed limit increases. In general, the probability of a fatal crash increases more than the probability of an injury crash. For crashes on rural State routes involving cars or light trucks and cars or light trucks, the information in table 3 can be interpreted as follows: A 1-percent increase in the posted speed limit is associated with an 11.9-percent increase in the probability of a fatal crash and a 1.32-percent increase in the probability of an injury crash. (Malyshkina and Mannering 2008) Due to the general increase in the probability of crashes on non-interstate highways, Malyshkina and Mannering (2008) recommended that, when considering potential increases in speed limits on rural roads, each road should be analyzed on a case-by-case basis.

Table 3. The increase in probability of a fatality and injury due to an increase in speed limit determined from accident-severity models. (Malyshkina and Mannering 2008, table 2)

Model	A 1% Speed Limit Increase is Associated With an X% Change in the Probability of a Fatal Crash	A 1% Speed Limit Increase is Associated With an X% Change in the Probability of an Injury Crash
Rural Country Road (C-LT) + (C-LT)	1.61	1.20
Rural Country Road (C-LT) + (HT)	2.77	2.35
Rural Country Road One vehicle	0.24	0.19
Rural State Route (C-LT) + (C-LT)	11.9	1.32
Rural State Route (C-LT) + (HT)	5.79	5.36
Rural State Route One vehicle	3.34	—
Rural City Street (C-LT) + (C-LT)	1.46	1.12
Rural City Street (C-LT) + (HT)	—	—
Rural City Street One vehicle	—	—
Rural U.S. Route (C-LT) + (C-LT)	—	—
Rural U.S. Route (C-LT) + (HT)	3.12	2.28
Rural U.S. Route One vehicle	—	—

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C = cars; LT = light trucks; HT = heavy trucks; (C-LT) + (C-LT) = crash involving a car or light truck and a car or light truck.
—No data.

In summary, higher operating speeds result in crashes that are more severe. If pedestrians are involved in crashes, the fatality risk increases as the impact speed increases. (Rosén et al. 2011) The published literature suggests that, for rural roads, the probability of fatal or injury crashes increases as the posted speed limit increases. (Malyshkina and Mannering 2008)

CRASH FREQUENCY–SPEED RELATIONSHIPS

While the relationship between crash severity and speed is well understood, the relationship between crash frequency and speed is less clear. Intuitively, "if on a road the driven speeds

become higher, the crash rate will also increase." (SWOV 2009) However, the relationship between speed and crash frequency is unknown.

Research by Kockelman et al. (2006) related crash rates and crash frequency to posted speed limits using statistical models. The study found that "crash rates rise with increasing speed limit, but at a decreasing rate," indicating that the increase in crash rates at higher posted speed limits is less than the increase in crash rates at lower posted speed limits. (Kockelman et al. 2006) Table 4 shows the relationship between higher posted speed limits, average driving speed, and total crash counts based on data from Washington State, Southern California, and Austin, Texas. As shown, the total crash count increases as the speed limit increases, and subsequently, the average driving speeds increase. (Kockelman et al. 2006)

Table 4. Safety effects of speed increases. (Kockelman et al. 2006)

Increase in Speed Limit (mph (km/h))	Change in Average Driving Speed (mph (km/h))	Change in Total Crash Count (%)
55 (88.6) to 65 (104.7)	+3 (4.8)	+3.3
65 (104.7) to 75 (120.8)	+3 (4.8)	+0.64

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Note: Calculations assume average high-speed roadway geometry.

Farmer (2016) reported similar findings to those of Kockelman et al. (2006) in a recent study. The author found that fatality rates and risk increased as maximum speed limits were raised in 41 States. The results indicated that "a 5 mph [8.1 km/h] increase in the maximum state speed limit was associated with an eight percent increase in fatality rates on interstates and freeways and a four percent increase on other roads." (Farmer 2016)

While studies have revealed the relationship between operating speeds and crash frequency, the exact relationship has not been well quantified. According to Transportation Research Board (TRB) *Special Report 254*, "speed is also linked to the probability of being in a crash, although the evidence is not as compelling because crashes are complex events that seldom can be attributed to a single factor." (TRB 1998)

In addition to research on increasing posted speed limits, Donnell et al. (2016) analyzed the effects of setting speed limits lower than engineering recommendations, which are typically set such that they are nearly equal to the 85th-percentile operating speed. The difference between the posted speed limit and the engineering recommended speed limit ranged from 5 to 25 mph (8.1 to 40.3 km/h) at various sites in Montana. The results of the study found that setting the speed limit 5 mph (8.1 km/h) below the recommended engineering value was associated with fewer total and fatal and injury crashes, while setting the posted speed limit 10 mph (16.1 km/h) lower than the recommended engineering value resulted in an increase in fatal plus injury crashes and a reduction in total crash frequency. A statistically significant association between posting speed limits 15 to 25 mph (24.2 to 40.3 km/h) lower than the engineering recommended could not be determined. (Donnell et al. 2016)

A consistent, quantifiable relationship between operating speed and crash frequency has not been reported in the literature. However, research suggests that changing the posted speed limit is associated with changes in the expected crash frequency. Increasing the posted speed limit has resulted in an increase in the expected crash frequency, while reducing the posted speed limit has shown mixed results in past research. Moreover, setting artificially lower posted speed limits may increase the variability in operating speeds among vehicles using two-lane rural highways, which may be associated with higher crash frequencies.

SAFETY-SPEED VARIANCE RELATIONSHIPS

Speed variance often refers to the difference in operating speeds among vehicles on a roadway. Separate studies by Solomon (1964) and Cirillo (1968) concluded that, as vehicle speeds deviated from the average speed of the traffic stream, crash involvement rates increased. Solomon (1964) noted that "as speeds departed from the average speed in either direction, the involvement rate increased in a nearly symmetrical fashion." The data from Solomon (1964) and Cirillo (1968) are shown in figure 4. The nighttime and daytime curves were developed by Solomon (1964), and the freeway curve was developed by Cirillo (1968).
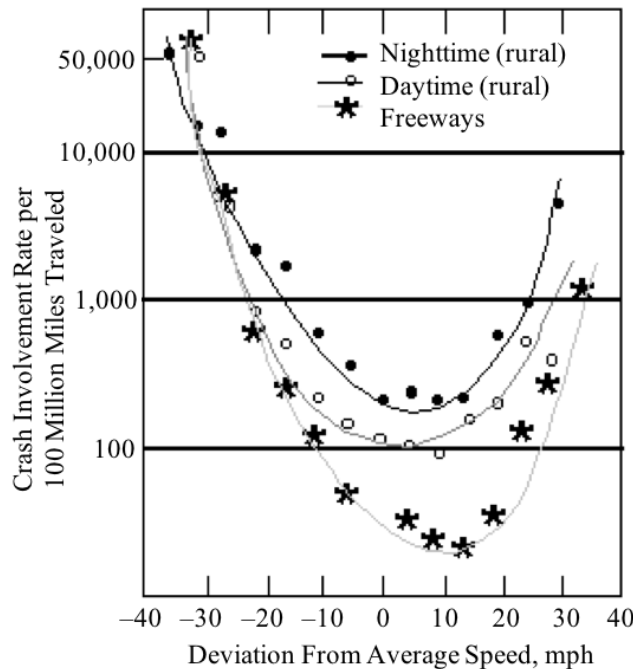
©Stuster et al. (figure 1).
1 mph = 1.60934 km/h.

Figure 4. Graph. Crash involvement rate as speed deviates from average travel speed from studies by Solomon (1964) and Cirillo (1968). (Stuster et al. 1998, figure 1)

Studies throughout the 1970s produced findings consistent with the research of Solomon (1964) and Cirillo (1968) with regard to speed deviation and crash severity. For example, the Research Triangle Institute, in 1970, studied crashes that occurred on roads with speed limits of 40 mph or higher. (TRB 1998) The results, which were similar to those found in Solomon (1964) and Cirillo

(1968), indicated that crash involvement increases as deviation from the average speed increases; however, the magnitude was not as large as found in the previous studies. (TRB 1998)

Similarly, West and Dunn in 1971 attempted to reproduce Solomon's curve, while excluding intersection data from the sample. The resulting curve was flatter than Solomon's, but it showed that, as speed deviations increase, the crash involvement rate increases. (TRB 1998)

Lave, in 1985, and Garber and Gadiraju, in 1988, produced similar findings. Both studies determined that speed dispersion and crash rates are correlated. Lave (1985) concluded that "speed dispersion significantly related to fatality rates for rural Interstates and rural and urban arterials," and Garber and Gadiraju (1988) found that "crash rates increased with increasing speed variance on all road classes." (TRB 1998)

In contrast to the previously determined U-shaped curves, Fildes et al., in 1991, determined a linear relationship between deviations from average travel speeds and crash involvement rates. (TRB 1998)

Davis (2002) concluded that "such positive correlations can be expected in situations where individual crash risk is either an increasing, or a decreasing, or a U-shaped function of speed, and so the correlations in themselves provide no evidence concerning the relation between speed and crash risk for individuals," and that although "such correlations can be expected in circumstances where individual risk is independent of speed variance, observation of these correlations provides no support for the hypothesis that increases in speed variance increase individual risk." In other words, Davis (2002) concluded that the study by Solomon (1964) and other studies by researchers who noted similar speed deviation-crash involvement rate relationships were ecological fallacies because individual crash risk based on the speed dispersion among a group of vehicles in the traffic stream does not clearly distinguish between individual and group risk measures.

SUMMARY

Based on the findings of the studies summarized in this guidance report, a relationship exists between speed and safety on moderate- and high-speed rural highways. For crash severity, higher vehicle operating speeds are associated with more severe crash outcomes. However, the relationship between crash frequency and speed is not as clear. There is some indication that increasing posted speed limits is associated with an increase in expected crash frequency; however, the relationship between operating speed and crash frequency has yet to be well established.

When applying the self-enforcing design concepts described in chapter 4 of this report, figure 2 and figure 3 may be used to predict how the expected frequency of various crash types may change. To do so, either observed or expected operating speeds for two different conditions can be used to compare the expected safety performance of the two different conditions.

CHAPTER 3. RELATIONSHIP BETWEEN SPEED AND GEOMETRIC DESIGN

Geometric roadway design practices in the United States rely on design controls and criteria set forth in the American Association of State Highway and Transportation Officials' (AASHTO) *A Policy on Geometric Design of Highways and Streets*, also known as the "Green Book." The design speed is defined in the Green Book as "the selected speed used to determine the various geometric features of the roadway." (AASHTO 2011) The Green Book either explicitly or implicitly uses the design speed concept to establish horizontal alignment, vertical alignment, and cross-section design elements. Examples include radius of curvature (R), stopping sight distance (SSD), braking distance (d_b), horizontal sight line offset (HSO), length of vertical curvature (L), maximum superelevation (e_{max}), maximum side friction factor (f_{max}), and lane and shoulder widths.

For the purposes of this guidance report, the designated design speed of a roadway is the speed established as part of the geometric design process. (Donnell et al. 2009) This speed is used to establish the geometric design criteria noted above and is equivalent to the design speed term used in the Green Book. The inferred design speed, which Donnell et al. (2009) defined as "the maximum speed for which all critical design-speed-related criteria are met at a particular location," is equivalent to the designated design speed when either minimum or limiting values of design criteria are used. However, the Green Book recommends using design values that exceed minimum values, and in such cases, the inferred design speed will exceed the designated design speed.

Operating speed models have often been used to assess geometric design consistency, most notably on two-lane rural highways. Many studies have estimated statistical models to predict vehicle operating speeds that may be used to evaluate highway design consistency. In many of the models, variables such as roadway geometric features, posted speed limit, and annual average daily traffic (AADT) can be input into the models to determine the vehicle operating speed under free-flow conditions (e.g., vehicle headways of 4 or more sec). While the most common speed output from these models is the 85th-percentile speed, statistical models of mean speed and the standard deviation of speed exist. Applying operating speed models may confirm that designated design speeds, posted speed limits, and driver expectations will all be more consistent when the roadway geometry is designed to manage speeds. (TRB 1998)

The design speed concept does not necessarily guarantee design consistency. The Green Book recommends minimum or limiting values for many speed-based geometric-design elements. When the geometric design values are larger than minimum values, the result is a higher inferred speed, which may be associated with higher operating speeds. This may produce instances where operating speeds on adjacent roadway segments are large or instances when the operating speed differs significantly from the designated design speed used to establish the geometric design features of the roadway. A more detailed explanation of design consistency can be found in later sections of this report.

This chapter examines the relationship between speed and geometric design. The different elements of geometric design, such as horizontal alignment, vertical alignment, and cross-section design elements, are related to speed. The chapter outlines how the designated design speed is

related to horizontal- and vertical-curve design criteria, the criteria for selecting cross-section elements, such as lane width, and the relationship between the designated design speed and inferred design speed. The geometric design features of a roadway subsequently influence operating speeds. In addition to discussing how geometric elements are associated with the designated design speed, this chapter describes various operating speed models that have been reported in the literature; this includes mean speed, speed dispersion, and 85th-percentile operating speed. There are examples of how to use operating speed prediction models to evaluate how geometric elements and other roadway characteristics affect driver speed choice. In addition to the illustrative examples shown in this chapter, other speed prediction models are shown in appendix A.

RELATIONSHIP BETWEEN DESIGNATED DESIGN SPEED AND GEOMETRIC DESIGN CRITERIA

Horizontal curve design is governed by the point–mass model, which prescribes a minimum radius of curvature as a function of the designated design speed, maximum superelevation of the roadway, and maximum side friction factor. (AASHTO 2011) The friction factor used in the geometric design of highways and streets is a demand value that is based on driver comfort thresholds rather than the side friction supply at the tire–pavement interface. The Green Book recommends limiting values for superelevation and side friction factor for horizontal-curve design based on the designated design speed. The radius of curvature equation found in the Green Book is shown in figure 5.

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

Figure 5. Equation. Radius of curvature. (AASHTO 2011)

Where:
R_{min} = minimum radius of curvature (ft (m)).
V = design speed (mph (km/h)).
e_{max} = maximum rate of roadway superelevation (percent).
f_{max} = maximum side friction (demand) factor.

Another fundamental geometric design criterion is the SSD, which is the distance needed for a driver to see an object on the roadway in front of the vehicle, react to it, and brake to a complete stop. The SSD is composed of two measures: (1) the distance traveled during perception–reaction time, and (2) the distance traveled during braking. Minimum SSD criteria are based on the assumptions that drivers travel at a speed equal to or below the designated design speed.

The braking distance in the SSD model (criteria) is determined by the formula shown in figure 6, assuming a level vertical grade.

$$d_b = 1.075\frac{V^2}{a}$$

Figure 6. Equation. Braking distance for level vertical grade. (AASHTO 2011)

Where:
    d_b = braking distance (ft (m)).
    a = deceleration rate (ft/s^2 (m/s^2)).

In cases where a vertical grade exists, the braking distance is modified as shown in figure 7:

$$d_b = \frac{V^2}{30\left[\left(\frac{a}{32.2}\right) \pm G\right]}$$

Figure 7. Equation. Braking distance when vertical grade exists. (AASHTO 2011)

Where G is the grade (rise/run, ft/ft (m/m)).

The braking distance is included as a part of the SSD along with the distance traveled during perception–reaction. The formula shown in figure 8 is used to determine minimum SSD criteria in the Green Book.

$$SSD = 1.47Vt + 1.075\frac{V^2}{a}$$

Figure 8. Equation. SSD. (AASHTO 2011)

Where t is brake reaction time (2.5 s).

Objects located along the inside of horizontal curves may pose a visual sight obstruction, which is also considered in horizontal-curve design. (AASHTO 2011) This is assessed using the HSO, which is determined as follows in figure 9.

$$HSO = R\left[1 - \cos\left(\frac{28.65S}{R}\right)\right]$$

Figure 9. Equation. HSO. (AASHTO 2011)

Where:
    S = stopping sight distance (ft (m)).
    R = radius of curve (ft (m)).

MEAN SPEED AND SPEED DISPERSION

While there are numerous statistical models that estimate or predict 85th-percentile operating speeds as a function of geometric design features, as shown in the next section, few models are available to predict mean operating speeds. The mean speed can be used to estimate the 85th-percentile speed, if speeds are normally distributed, by adding the standard deviation of speed to the mean speed. (Roess et al. 2011) This enables the opportunity to assess the association between speed dispersion and geometric design features in a statistical model. This section of the guidance report shows several examples of statistical models that include mean speed and speed dispersion metrics as a function of geometric design features. In each case, the speed metric (i.e., posted speed limit, mean speed, or standard deviation of speed) is the dependent variable, while roadway

geometric features and other site-specific features are the independent variables in the model. All of the models are linear models, where the dimension of the independent variable is multiplied by a regression coefficient to determine how the roadway design features influences the expected speed metric. Several other statistical models of vehicle operating speeds are shown in appendix A.

Himes et al. (2011) used a system of linear equations to estimate models for the posted speed limit, mean speed, and standard deviation of speed. An interpretation of the models is provided in table 5. Data were collected on urban and rural two-lane undivided highways in Virginia and Pennsylvania. These data included roadway characteristics, vehicle operating speeds, and hourly traffic flow rates. An example of a typical linear model used by Himes et al. (2011) for their system of simultaneous equations is shown in figure 10. The linear model is used with the information provided in table 5.

$$y = \alpha + \sum \beta X$$

Figure 10. Equation. Typical linear model.

Where:
    y = speed measure (posted speed limit, mean speed, or speed deviation).
    α = intercept for posted speed limit, mean speed, or speed deviation equation.
    β = coefficient for road characteristics.
    X = road characteristics (geometric features, hourly traffic volume, etc.).

Through the system of equations, the authors could determine the relationship between roadway and roadside features, and traffic flow on posted speed limit, mean speed, and standard deviation. The study found that an increase in posted speed limit and shoulder width was associated with an increase in mean speed. Additionally, Himes et al. (2011) concluded that hourly traffic volume, vertical grade, wooded adjacent land use, and left-hand horizontal curves were negatively associated with speed deviation. The proportion of heavy vehicles was positively correlated with speed deviation. (Himes et al. 2011) Although the simultaneous equations are not shown in this report, the relationship between the dependent and independent variables from the Himes et al. (2011) study are described in table 5 and used in conjunction with the typical linear model shown in figure 10. For example, a 1-ft (0.3-m) increase in the total shoulder width is associated with a 0.33-ft (0.1-m) increase in the expected mean operating speed. These effect sizes are applicable to the range of independent variables included in the sample used to estimate the operating speed models. Readers interested in reviewing the results of the research are encouraged to review the Himes et al. (2011) study.

Table 5. Interpretation of the mean speed and speed deviation model produced by Himes et al. (2011).

Dependent Variable (mph (km/h))	Independent Variable	Effect Size	Interpretation
Mean speed	Total shoulder width (ft (m))	0.33	A 1-ft (0.3-m) increase in total shoulder width is associated with a 0.33-mph (0.5 km/h) increase in mean speed.
Mean speed	Number of access points within 1,000 ft (305 m) of location	–0.29	A one-unit increase in the number of access points within 1,000 ft (305 m) of the speed measurement location is associated with a 0.29-mph (0.5-km/h) decrease in mean speed.
Mean speed	Presence of median or two-way left-turn lane (1 if present; 0 otherwise)	–3.22	Presence of a median or turning lane is associated with a 3.22-mph (5.2-km/h) decrease in mean speed.
Mean speed	Presence of at-grade rail crossing within 500 ft (152 m) (1 if present; 0 otherwise)	–5.64	Presence of an at-grade rail crossing within 500 ft (152 m)of the speed measurement location is associated with a 5.64-mph (9.1-km/h) decrease in mean speed.
(Mean speed	Left-hand curve indicator (1 if left-hand curve; 0 otherwise)	–1.41	A left-hand curve is associated with a 1.41-mph (2.3-km/h) decrease in mean speed.
Mean speed	Crest vertical curve indicator (1 if crest vertical curve; 0 otherwise)	–1.18	A crest vertical curve is associated with a 1.18-mph (1.9-km/h) decrease in mean speed.
Speed deviation	Hourly traffic volume (vph)	–0.01	A 1-vph increase in hourly traffic volume is associated with a 0.01-mph (0.02-km/h) decrease in speed deviation.
Speed deviation	Grade (%)	–0.08	A 1% increase in grade is associated with a 0.08-mph (0.13-km/h) decrease in speed deviation.
Speed deviation	Wooded adjacent land use indicator (1 if wooded; 0 otherwise)	–1.05	Wooded adjacent land use is associated with a 1.05-mph (1.7-km/h) decrease in speed deviation.
Speed deviation	Left-hand curve indicator (1 if left-hand curve; 0 otherwise)	–0.47	A left-hand curve is associated with a 0.47-mph (0.8-km/h) decrease in mean speed.
Speed deviation	Heavy vehicles in traffic stream (%)	0.05	A 1% increase in heavy vehicles in the traffic stream is associated with a 0.05-mph (0.1-km/h) increase in speed deviation.

vph = vehicles per hour.

Similarly, a study by Figueroa Medina and Tarko (2005) estimated statistical models that considered the combined effect of mean speed and speed deviation to predict percentile operating speeds. The free-flow speed models were developed for tangent segments and horizontal curves on two-lane rural highways. The data used to develop the ordinary least

squares (OLS) regression model were collected in Indiana and included roadway geometric design features, free-flow speeds, and sight distances. Statistical models were estimated for operating speeds on tangent sections and operating speeds on horizontal curves. The equation shown in figure 11 was developed to predict operating speeds on two-lane rural highway tangent sections.

$$
\begin{aligned}
V_p = \; & 57.137 - 0.071 \times TR - 3.082 \times PSL_{50} - 0.131 \times GR - 1.034 \times RES \\
& + 2.38 \times 10^{-3} \times SD - 1.67 \times 10^{-6} \times SD^2 - 0.422 \times INT + 0.040 \times PAV \\
& + 0.394 \times GSW + 0.054 \times USW - 2.233 \times FC + 5.982 \times Z_p \\
& + 1.428 \times \left(Z_p \times PSL_{50} \right) + 0.061 \times \left(Z_p \times GR \right) + 0.292 \times \left(Z_p \times INT \right) \\
& - 0.038 \times \left(Z_p \times PAV \right) - 0.012 \times (Z_p \times CLR)
\end{aligned}
$$

Figure 11. Equation. Model for speed on tangent roadway sections.

Where:
    V_p = speed on tangent section (mph (km/h)).
    TR = percentage of trucks.
    PSL_{50} = equal to 1 if the posted speed limit is 50 mph (80.5 km/h), equal to 0 if the posted
        speed limit is 55 mph (88.5 km/h).
    GR = highway grade (percent).
    RES = equal to 1 if the segment has 10 or more residential driveways per mile (1.6 km), 0
        otherwise.
    SD = available stopping sight distance (ft (m)).
    INT = equal to 1 if an intersection is located 350 ft (106.7 m) before or after the spot, 0
        otherwise.
    PAV = pavement width, includes the traveled way and both paved shoulders (ft (m)).
    GSW = total gravel shoulder width (ft (m)).
    USW = total untreated shoulder width (ft (m)).
    CLR = roadside clear zone, includes the total gravel and total untreated shoulders (ft (m)).
    FC = equal to 1 if the spot is located on a flat curve (radius larger than 1,700 ft (518.2 m)), 0
        otherwise.
    Z_p = standardized normal variable corresponding to a selected percentile.

The equation from the Figueroa Medina and Tarko (2005) study used to predict operating speeds on horizontal curves of two-lane rural highways is shown in figure 12.

$$
\begin{aligned}
V_p = \; & 47.664 + 3.44 \times 10^{-3} \times SD - 2.639 \times RES - 2.541 \times DC + 7.954 \times SE - 0.624 \times SE^2 \\
& + 4.158 \times Z_p + 0.236 \times \left(Z_p \times DC \right) - 0.199 \times \left(Z_p \times SE \right)
\end{aligned}
$$

Figure 12. Equation. Model for speed on horizontal curve roadway sections. (Figueroa Medina and Tarko 2005)

Where:
    V_p = speed on horizontal curve section (mph (km/h)).
    DC = degree of curvature (degrees).
    SE = maximum superelevation rate (percent).

The statistical models shown above consider several roadway characteristics and the posted speed limit to predict the free-flow vehicle operating speeds, which can be used in methods 1 through 4 of the self-enforcing roadway concepts shown in chapter 5. Certain variables in each equation are factors that affect the mean speed or standard deviation of speed. The degree of curvature and superelevation are factors for both mean speed and speed deviation. The variable in the equation containing Z_p is associated with the standard deviation. The Z-statistic, which reflects a value representative of a percentile value under the standard normal distribution, is shown in figure 11 and figure 12. This value can be used to predict the percentile speeds. For example, Z_{50} is equal to 0 for 50th-percentile speeds, and Z_{85} is equal to 1.036 for 85th-percentile speeds. The interpretations of the variables and parameters for the equations in figure 11 and are shown in table 6.

Table 6. Interpretation of the mean speed equations for curve and tangent segments (figure 11 and figure 12) produced by Figueroa Medina and Tarko (2005).

Model	Variable	Effect Size	Interpretation
Speed on tangent section (figure 11)	Trucks (%)	–0.071	A 1% increase in trucks is associated with a 0.071-mph (0.1-km/h) decrease in 85th-percentile speeds on tangent sections.
Speed on tangent section (figure 11)	Posted speed limit (1 if the posted speed limit is 50 mph (80.5 km/h); 0 if the posted speed limit is 55 mph (88.5 km/h)	–3.082	A posted speed limit of 50 mph (80.5 km/h) is associated with a 3.082-mph (5.0-km/h) decrease in 85th-percentile speeds on tangent sections as compared to a posted speed limit of 55 mph (88.5 km/h).
Speed on tangent section (figure 11)	Highway grade (%)	–0.131	A 1% increase in highway grade is associated with a 0.131-mph (0.2-km/h) decrease in 85th-percentile speeds on tangent sections.
Speed on tangent section (figure 11)	Residential driveways (1 if the segment has 10 or more residential driveways per mile ((1.6 km); 0 otherwise)	–1.034	Presence of 10 or more residential driveways per mile (1.6 km) is associated with a 1.034-mph (1.7-km/h) decrease in 85th-percentile speeds on tangent sections as compared to segments that contain less than 10 residential driveways per mile (1.6 km).
Speed on tangent section (figure 11)	Sight distance (ft (m))	2.38×10^{-3}	A 1-ft (0.3-m) increase in sight distance is associated with a 2.38×10^{-3}-mph (0.004-km/h) increase in 85th-percentile speeds on tangent sections.
Speed on tangent section (figure 11)	Sight distance squared (ft^2 (m^2))	1.67×10^{-6}	A 1-ft (0.3-m) increase in sight distance squared is associated with a 1.67×10^{-6}-mph (5.1×10^{-7}-km/h) increase in 85th-percentile speeds on tangent sections.
Speed on tangent section (figure 11)	Intersection (if an intersection is located 350 ft (106.7 m) before or after the spot; 0 otherwise)	–0.442	Presence of an intersection within 350 ft (106.7 m) of the spot is associated with a 0.442-mph (0.7-km/h) decrease in 85th-percentile speeds on tangent sections.

Model	Variable	Effect Size	Interpretation
Speed on tangent section (figure 11)	Pavement width; includes the traveled way and both paved shoulders (ft (m))	0.040	A 1-ft increase in pavement width is associated with a 0.040-mph increase in 85th-percentile speeds on tangent sections.
Speed on tangent section (figure 11)	Total gravel shoulder width (ft (m))	0.394	A 1-ft (0.3-m) increase in total gravel shoulder width is associated with a 0.394-mph (0.6-km/h) increase in 85th-percentile speeds on tangent sections.
Speed on tangent section (figure 11)	Total untreated shoulder width (ft (m))	0.054	A 1-ft (0.3-m) increase in total untreated shoulder width is associated with a 0.054-mph (0.1-km/h) increase in 85th-percentile speeds on tangent sections.
Speed on tangent section (figure 11)	Flat curve (1 if the spot is located on a flat curve (radius larger than 1,700 ft (518.2 m)); 0 otherwise)	–2.233	A flat curve is associated with a 2.233-mph (3.6-km/h) decrease in 85th-percentile speeds on tangent sections.
Speed on tangent section (figure 11)	Posted speed limit (1 if the posted speed limit is 50 mph (80.5 km/h); 0 if the posted speed limit is 55 mph (88.5 km/h)) x Z_p	1.428 x 1.036 = 1.479	A posted speed limit of 50 mph (80.5 km/h) is associated with a 1.479-mph (2.4-km/h) increase in 85th-percentile speeds on tangent sections as compared to a posted speed limit of 55 mph (88.5 km/h). A posted speed limit of 50 mph (80.5 km/h) is associated with a greater dispersion in operating speeds.
Speed on tangent section (figure 11)	Highway grade (%) x Z_p	0.061 x 1.036 = 0.063	A 1% increase in highway grade is associated with a 0.063-mph (0.1-km/h) increase in 85th-percentile speeds on tangent sections. A 1% increase in highway grade is associated with a greater dispersion in operating speeds.
Speed on tangent section (figure 11)	Intersection (if an intersection is located 350 ft (106.7 m) before or after the spot; 0 otherwise) x Z_p	0.292 x 1.036 = 0.303	Presence of an intersection within 350 ft (106.7 m) of the spot is associated with a 0.303-mph (0.5-km/h) increase in 85th-percentile speeds on tangent sections. Presence of an intersection within 350 ft (106.7 m) of the spot is associated with a greater dispersion in operating speeds.
Speed on tangent section (figure 11)	Pavement width, includes the traveled way and both paved shoulders (ft (m)) x Z_p	–0.038 x 1.036 = –0.039	A 1-ft (0.3-m) increase in pavement width is associated with a 0.039-mph (0.1-km/h) decrease in 85th-percentile speeds on tangent sections. A 1-ft (0.3 m) increase in pavement width is associated with less dispersion in operating speeds.
Speed on tangent section (figure 11)	Roadside clear zone, includes the total gravel and total untreated shoulders (ft (m)) x Z_p	–0.012 x 1.036 = –0.012	A 1-ft (0.3-m) increase in roadside clear zone is associated with a 0.012-mph (0.02-km/h) decrease in 85th-percentile speeds on tangent sections. A 1-ft (0.3-m) increase in roadside clear zone is associated with less dispersion in operating speeds.

Model	Variable	Effect Size	Interpretation
Speed on horizontal-curve section (figure 13)	Sight distance (ft (m))	3.44×10^{-3}	A 1-ft (0.3-m) increase in sight distance is associated with a 3.44×10^{-3}-mph (0.01-km/h) increase in 85th-percentile speeds on curve sections.
Speed on horizontal-curve section (figure 13)	Residential driveways (1 if the segment has 10 or more residential driveways per mile (1.6 km); 0 otherwise)	–2.639	Presence of 10 or more residential driveways per mile (1.6 km) is associated with a 2.639-mph (4.2-km/h) decrease in 85th-percentile speeds on curve sections as compared to segments that contain less than 10 residential driveways per mile (1.6 km).
Speed on horizontal-curve section (figure 13)	Degree of curvature (degrees)	–2.541	A 1-degree increase in degree of curvature is associated with a 2.541-mph (4.1-km/h) decrease in 85th-percentile speeds on curve sections.
Speed on horizontal-curve section (figure 13)	Maximum superelevation rate (%)	7.954	A 1% increase in maximum superelevation rate is associated with a 7.954-mph (12.8-km/h) increase in 85th-percentile speeds on curve sections.
Speed on horizontal-curve section (figure 13)	Maximum superelevation rate squared (%)	–0.624	A 1% increase in maximum superelevation rate squared is associated with a 0.624-mph (1.0-km/h) decrease in 85th-percentile speeds on curve sections.
Speed on horizontal-curve section (figure 13)	Degree of curvature (degrees) x Z_p	$0.236 \times 1.036 = 0.244$	A 1% increase in degree of curvature is associated with a 0.244-mph (0.4-km/h) increase in 85th-percentile speeds on curve sections. A 1-degree increase in degree of curvature is associated with a greater dispersion in operating speeds.
Speed on horizontal-curve section (figure 13)	Maximum superelevation rate (%) x Z_p	$-0.199 \times 1.036 = -0.206$	A 1% increase in maximum superelevation rate is associated with a 0.206-mph (0.3-km/h) decrease in 85th-percentile speeds on curve sections. A 1% increase in maximum superelevation rate is associated with less dispersion in operating speeds.

Drivers select operating speeds based on multiple factors, several of which include the roadway design features. The parameters shown in table 6 generally show that more restrictive geometrics and roadways that have built-up adjacent land use (such as residential and commercial developments) tend to be associated with lower operating speeds.

For tangent segments, the proportion of trucks in the traffic stream, posted speed limit less than 50 mph (80.5 km/h), highway grade, presence of residential driveways, presence of an intersection, and presence of a flat curve are associated with a decrease in mean speeds, while increasing sight distance, pavement width, gravel shoulder width, and untreated shoulder width are associated with an increase in mean speeds. Furthermore, on tangent roadway sections, increasing pavement width and roadside clear zone results in a decrease in speed dispersion, while speed limit, highway grade, and presence of an intersection are associated with an increase in speed dispersion.

For horizontal curves, increased sight distance and the superelevation rates are associated with an increase in mean speed, while presence of residential development, degree of curvature, and the superelevation rate squared is associated with a decrease in mean speed. Additionally, for horizontal curves, the degree of curvature is associated with an increase in speed dispersion, while the superelevation rate is associated with a decrease in speed dispersion.

An example using the models provided by Figueroa Medina and Tarko (2005) and Himes et al. (2011) that estimate mean speed and speed dispersion/deviation is shown in table 7 through table 12. Figueroa Medina and Tarko (2005) determined two distinct models for mean speed and speed dispersion: one for tangent segments, one for horizontal curves. Both are shown in the tables. These examples illustrate how to apply operating speed models to predict driver speed choice on two-lane rural highways. Operating speed prediction models may be used in methods 1 through 4 of the self-enforcing roadway design concepts presented in chapter 5. In table 7 through table 12, the coefficient is multiplied by the dimension to produce a mean speed estimate associated with the dimensions. All these associations are added to produce the predicted mean speed on tangent- or horizontal-curve segments.

Table 7. Example using the tangent segments mean speed model from Figueroa Medina and Tarko (2005).

Variable	Coefficient	Value or Dimension	Mean Speed (Coefficient x Value) (mph (km/h))
Constant	57.137	—	57.137 (92.0)
Percent of trucks, *TR*	–0.071	10	–0.71 (–1.1)
50-mph speed limit indicator, PSL_{50} (1 if *PSL* is 50 mph (80.5 km/h); 0 if *PSL* is 55 mph (88.5 km/h)	–3.082	0	0 (0)
Highway grade, *GR* (%)	–0.131	2.28	–0.299 (–0.5)
Residential development indicator, *RES* (1 if segment has 10 or more residential driveways per mile; 0 otherwise)	–1.034	0	0 (0)
Sight distance, *SD* (ft (m))	0.00238	1290	3.070 (4.9)
Sight distance squared, SD^2 (ft^2 (m^2))	–0.0000017	1,664,100	–2.779 (–4.5)
Intersection indicator, *INT* (1 if an intersection is located 350 ft (106.7 m) before or after the spot; 0 otherwise)	–0.422	0	0 (0)
Pavement width, *PAV* (ft (m))	0.040	30	1.20 (1.9)
Gravel shoulder width, *GSW* (ft (m))	0.394	0	0 (0)
Untreated shoulder width, *USW* (ft (m))	0.0544	0	0 (0)
Flat curve indicator, *FLC* (1 if spot is located on a flat curve (radius larger than 1,700 ft (518.2 m), 0 otherwise)	–2.233	0	0 (0)
Predicted Mean Operating Speed (mph (km/h))	—	—	**57.6 (92.7)**

—Not applicable.

Table 8. Example using the horizontal curves mean speed model from Figueroa Medina and Tarko (2005).

Variable	Coefficient	Value or Dimension	Mean Speed (Coefficient x Value) (mph (km/h))
Constant	47.664	—	47.664 (76.7)
Sight distance, *SD*	0.003	1,290	3.87 (6.2)
Residential development indicator, *RES*	–2.639	0	0 (0)
Degree of curvature, *DC* (%)	–2.541	8	–20.328 (–32.7)
Superelevation rate, *SE* (%)	7.954	6.6	52.496 (84.5)
Superelevation rate squared, SE^2	–0.624	43.56	–27.181 (–43.7)
Predicted Mean Operating Speed (mph (km/h))	—	—	**56.5 (90.9)**

—Not applicable.

Table 9. Example using the mean speed model from Himes et al. (2011).

Variable	Coefficient	Value or Dimension	Mean Speed (Coefficient x Value) (mph (km/h))
Constant	18.2	—	18.2 (29.3)
Posted speed limit (mph (km/h))	0.6	55.65	33.39 (53.7)
Total shoulder width (ft (m))	0.33	8	2.64 (4.2)
No. access pts within 1,000 ft (m) of collection location	–0.29	0	0 (0)
Presence of median or turn lane (1 presence; 0 other)	–3.22	0	0 (0)
Presence of at-grade rail crossing within 500 ft (152.4 m) (1 presence; 0 other)	–5.64	0	0 (0)
Left-hand curve indicator (1 if left-hand curve; 0 otherwise)	–1.41	1	–1.41 (–2.3)
Crest vertical curve indicator (1 if crest vertical curve; 0 otherwise)	–1.18	0	0 (0)
Predicted Mean Operating Speed (mph (km/h))	—	—	**52.8 (85.0)**

—Not applicable.

Table 10. Example using the tangent segments speed dispersion/deviation model from Figueroa Medina and Tarko (2005).

Variable	Coefficient	Value or Dimension	Speed Dispersion/ Deviation (Coefficient x Value) (mph (km/h))
Constant, Z_p	5.9816	—	6.197 (10.0)
50-mph (80.5 km/h) speed limit indicator, $Z_p\text{-}PSL_{50}$	1.428	0	0 (0)
Highway grade, $Z_p\text{-}GRA$	0.061	2.28	0.144 (0.2)
Intersection indicator, $Z_p\text{-}INT$	0.292	0	0
Pavement width, $Z_p\text{-}PAV$	–0.038	30	–1.181 (–1.9)
Roadside clear zone, $Z_p\text{-}CLR$ (roadside clear zone, includes the total gravel and total untreated shoulders (ft (m)))	–0.012	8	–0.099 (–0.2)
Predicted Speed Dispersion (mph (km/h))	—	—	**5.1 (8.2)**

—Not applicable.

Table 11. Example using the horizontal curves speed dispersion/deviation model from Figueroa Medina and Tarko (2005).

Variable	Coefficient	Value or Dimension	Speed Dispersion/ Deviation (Coefficient x Value) (mph (km/h))
Constant, Z_p	4.158	—	4.308 ()
Degree of curvature, $Z_p\text{-}DC$	0.236	8	1.956
Superelevation rate, $Z_p\text{-}SE$	–0.199	6.6	–1.360
Predicted Speed Dispersion (mph (km/h))	—	—	**4.9 (6.9)**

—Not applicable.

Table 12. Example using the speed dispersion/deviation model from Himes et al. (2011).

Variable	Coefficient	Value or Dimension	Speed Dispersion/ Deviation (Coefficient x Value) (mph (km/h))
Constant	7.45	—	7.45 (12.0)
Posted speed limit (mph (km/h))	0.1	55.65	5.56 (8.9)
Mean speed (mph (km/h))	–0.09	52.82	–4.75 (–7.6)
Hourly traffic volume (vph)	–0.01	104.17	–1.04 (–1.7)
Grade (%)	–0.08	2.28	–0.18 (–0.3)
Wooded adjacent land use indicator (1 if wooded; 0 otherwise)	–1.05	1	–1.05 (–1.7)
Left-hand curve indicator (1 if left-hand curve; 0 otherwise)	–0.47	1	–0.47 (–0.8)
Heavy vehicles in traffic stream (%)	0.05	10	0.5 (0.8)
Predicted Speed Dispersion (mph (km/h))	—	—	**6.0 (9.7)**

—Not applicable.
vph = vehicles per hour.

As shown in table 7 through table 12, Figueroa Medina and Tarko (2005) and Himes et al. (2011) use different variables to predict mean speeds and speed dispersion. Using the models by Figueroa Medina and Tarko (2005), the mean speed was predicted to be 57.6 mph (92.7 km/h) for tangent segments and 56.5 mph for horizontal curves. Differently, the predicted mean speed using the Himes et al. (2011) model was 52.8 mph (85.0 km/h). Using the models by Figueroa Medina and Tarko (2005), the speed dispersion was predicted to be 5.1 mph (8.2 km/h) for tangent segments and 4.9 mph (7.9 km/h) for horizontal curves. The speed dispersion predicted using the Himes et al. (2011) model was 6.0 mph (9.7 km/h), which is similar to the results from Figueroa Medina and Tarko (2005). One possible explanation for the discrepancies in the results might be the use of different variables across the models. Additionally, the Figueroa Medina and Tarko (2005) models separate tangent segments and horizontal curves.

85TH-PERCENTILE SPEED

The 85th-percentile speed represents the speed at which 85 percent of vehicles are traveling at or below under free-flow conditions. This value can be used to establish posted speed limits, as recommended by the *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) or to evaluate the design consistency of a roadway. (FHWA 2009) Numerous studies have estimated linear regression models to predict 85th-percentile speeds on horizontal curves and tangents. Several geometric design features as well as the posted speed limit have been included in the speed-prediction models. A summary of these models for two-lane rural highways is provided below. Like the mean and speed dispersion models shown in the previous section of this guidance report, 85th-percentile operating-speed-prediction models can be used to estimate driver speed choice in methods 1 through 4 of the self-enforcing roadway design concepts presented in chapter 5.

Speed Prediction Models

Krammes et al. (1995) collected speed and geometric design data along horizontal curves and approach tangents in five States. The data were used to develop a model to predict operating speeds on both curves and approach tangents, and these models were then used to evaluate design consistency between successive geometric features. The geometric features included in the regression models of 85th-percentile operating speed were the degree of curvature, length of curvature, deflection angle, and in some cases, the 85th-percentile speed on approach tangents. The study determined that an increase in the degree of curvature and deflection angle results in a decrease in 85th-percentile speeds on the curve. For curves less than or equal to 4 degrees, as the length of the curve increases, the 85th-percentile speeds on the curve increase, while for curves greater than 4 degrees, as the length of the curve increases, the 85th-percentile operating speeds on the curve decrease. Additionally, as the 85th-percentile speed on the approach tangent increases, the 85th-percentile operating speeds on the curve increase. The equations developed from this study are shown in appendix A.

Fitzpatrick et al. (2000a) collected data on two-lane rural highways in several States to predict the 85th-percentile speed of passenger cars. The 85th-percentile operating speed models are shown in table 33 in appendix A and include the radius of curvature and the rate of vertical curvature. The radius of curvature was found to be the best predictor of operating speeds for horizontal curves on grade, while the rate of vertical curvature was found to be the best indicator of operating speeds on vertical curves that are present on horizontal tangent sections. (Fitzpatrick et al. 2000a) It was determined that the radius of curve significantly affects the 85th-percentile operating speeds on horizontal alignments. When the radius of curve is approximately 820 ft (250 m), 85th-percentile operating speeds decrease sharply, while 85th-percentile speeds on curves with a radius of approximately 2,625 ft (800 m) are similar to the 85th-percentile operating speed on long tangents. (Fitzpatrick et al. 2000a)

Similar to Fitzpatrick et al. (2000a), Misaghi and Hassan (2005) developed models to predict the 85th-percentile operating speed on horizontal curves by considering the radius of curve. Data were collected along 20 horizontal curves of two-lane rural highways in Canada; the data were used to analyze geometric design consistency. (Misaghi and Hassan 2005) Statistical models to predict the speed differential between the approach tangent and the horizontal curve were estimated. The equations developed by Misaghi and Hassan (2005) are shown in appendix A. The study found that an increase in the radius of curvature resulted in an increase in the 85th-percentile speed at the midpoint of the curve and a decrease in the 85th-percentile speed differential. It was also found that as the speed on the approach tangent increases, the deflection angle of circular curve increases, the shoulder width decreases, and the vertical grade increases, the 85th-percentile speed differential also increases. However, as the shoulder width increases, the 85th-percentile speed differential decreased.

Fitzpatrick et al. (2005) and Fitzpatrick et al. (2003) used the posted speed limit on tangent sections of two-lane rural highways to determine the 85th-percentile operating speed using linear regression equations. Both studies determined that geometric design features, including access density and parking along the street, are associated with 85th-percentile operating speeds. The authors also found that the posted speed limit is highly correlated with the 85th-percentile operating speed. (Fitzpatrick et al., 2005) Access density and the presence of parking were

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negatively correlated with operating speeds. Multiple 85th-percentile speed models were developed for the various road types and included the posted speed limit (i.e., there were separate models for suburban/urban arterial, suburban/urban collector, suburban/urban local, and rural arterial roads). The model developed for rural arterial roadways showed a positive relationship between estimated 85th-percentile operating speeds and the posted speed limit. A 1-mph (1.6-km/h) increase in posted speed limit was associated with a 0.517-mph (0.8-km/h) increase in 85th-percentile speeds for rural arterials. (Fitzpatrick et al. 2003) The equations from Fitzpatrick et al. (2005) and Fitzpatrick et al. (2003) are shown in appendix A.

Schurr et al. (2002) used data collected on rural two-lane highways in Nebraska to predict the 85th- and 95th-percentile operating speeds on rural two-lane highways, which were used to assess design consistency. (Schurr et al. 2002) The speed prediction equations included independent variables such as deflection angle, length of horizontal curve, approach grade, and average daily traffic. The 85th- and 95th-percentile operating speed equations are shown in appendix A. The study concluded that drivers tend to increase operating speeds as the curve is lengthened and 85th-percentile operating speeds decrease as the grade increases. (Schurr et al. 2002)

Lane and shoulder width along with radius of curvature can also affect operating speeds. Lamm and Choueiri (1987) used these variables to develop operating speed prediction models for horizontal curves. Separate regression equations were estimated based on the lane width, which ranged from 10 to 12 ft (3.0 to 3.7 m). "Good" designs were shown to have degree of curvature changes of 5 degrees or less between geometric elements, 85th-percentile speeds that vary less than or equal to 6 mph (9.7 km/h), and radii greater than or equal to 1,200 ft (365.8 m), while "poor" designs had degree of curvature changes larger than 10 degrees, 85th-percentile speeds that vary by more than 12 mph (19.3 km/h), and curve radii less than 600 ft (182.3 m). (Lamm and Choueiri 1987, Lamm et al. 1988) The thresholds for "good" and "poor" designs were based on accident data. Lamm and Choueiri (1987) noted that the average annual daily traffic had little influence on the estimated 85th-percentile operating speed of drivers.

On low-speed, two-lane rural highways in Australia, McLean (1979) estimated OLS linear regression models to predict 85th-percentile operating speeds using variables that included the desired 85th-percentile speed and the curve radius. Comparable to previous studies, it was determined that the curve radius influences the 85th percentile and desired speed of drivers. (McLean 1979) The study determined that an increase in the desired speed is associated with an increase in the 85th-percentile operating speed, and an increase in the inverse curve radius is associated with a decrease in the 85th-percentile operating speed. The 85th-percentile operating speed models from McLean (1979) are shown in appendix A.

While the majority of studies previously described focused on high-speed, two-lane rural roads, Banihashemi et al. (2011) developed operating speed prediction models for low-speed, rural two-lane highways. The posted speed limit ranged from 25 to 40 mph (40.2 to 64.4 km/h). The study estimated regression models to predict 85th-percentile operating speeds on tangents and horizontal curves. One statistical model calculated the 85th-percentile operating speed on a tangent section of roadway using the radius of the preceding curve and the posted speed limit, while another model predicted the 85th-percentile operating speed on a tangent section using the posted speed limit, roadside hazard rating, and the length of the tangent. Additionally,

Banihashemi et al. (2011) predicted operating speeds on curves using the radius of curvature. Posted speed limit and length of tangent were found to have a positive association with operating speeds. The radius of the preceding curve, roadside hazard rating, and radius of the subject curve were found to have a negative association with operating speeds. The 85th-percentile operating speed models from Banihashemi et al. (2011) are shown in table 33 in appendix A. These models are also incorporated into the Federal Highway Administration (FHWA) Interactive Highway Safety Design Model (IHSDM) Design Consistency Module (DCM). (FHWA 2016a)

The TRB's *Transportation Research Circular E-C151* indicated there is a lack of uniformity between models to predict 85th-percentile operating speeds. (TRB 2011) This can be attributed to the sheer number of models available and the use of many different predictor variables. The circular also states that horizontal curve radius is the only statistically significant variable affecting 85th-percentile operating speeds on alignments containing a horizontal curve. (TRB 2011)

HORIZONTAL ALIGNMENT AND SPEED RELATIONSHIP

A relationship between the horizontal alignment of a roadway and operating speed is well established. The following section describes the design process for horizontal alignment features and explains how operating speeds are affected by horizontal alignment design features.

Relationship Between Radius of Curvature and Speeds

To compare the relationship between radius of curvature and 85th-percentile operating speed, equations 1 and 2 by Misaghi and Hassan (2005), equations 1–4 by Fitzpatrick et al. (2000a), and equation 1 by McLean (1979), shown in table 33 in appendix A, from the studies reviewed previously, were used. The resulting plot is shown in figure 13. The vertical axis shows the 85th-percentile operating speeds, while the horizontal axis shows the radius of curvature. Equation 1 from McLean (1979) included the desired speed of the 85th-percentile car (V_F) and the curve radius to determine the 85th-percentile speed. The desired speed of the 85th-percentile car is the speed which cars desire to travel based on alignment characteristics of a roadway, including topography, cross section, adjacent land use, and traffic volumes. (McLean, 1979) To accommodate this, the equation was plotted using three different values for V_F. The design speed for a given maximum rate of superelevation-minimum radius combination is also shown in figure 13, which is based on table 3-7 of the AASHTO Green Book. (AASHTO 2011)
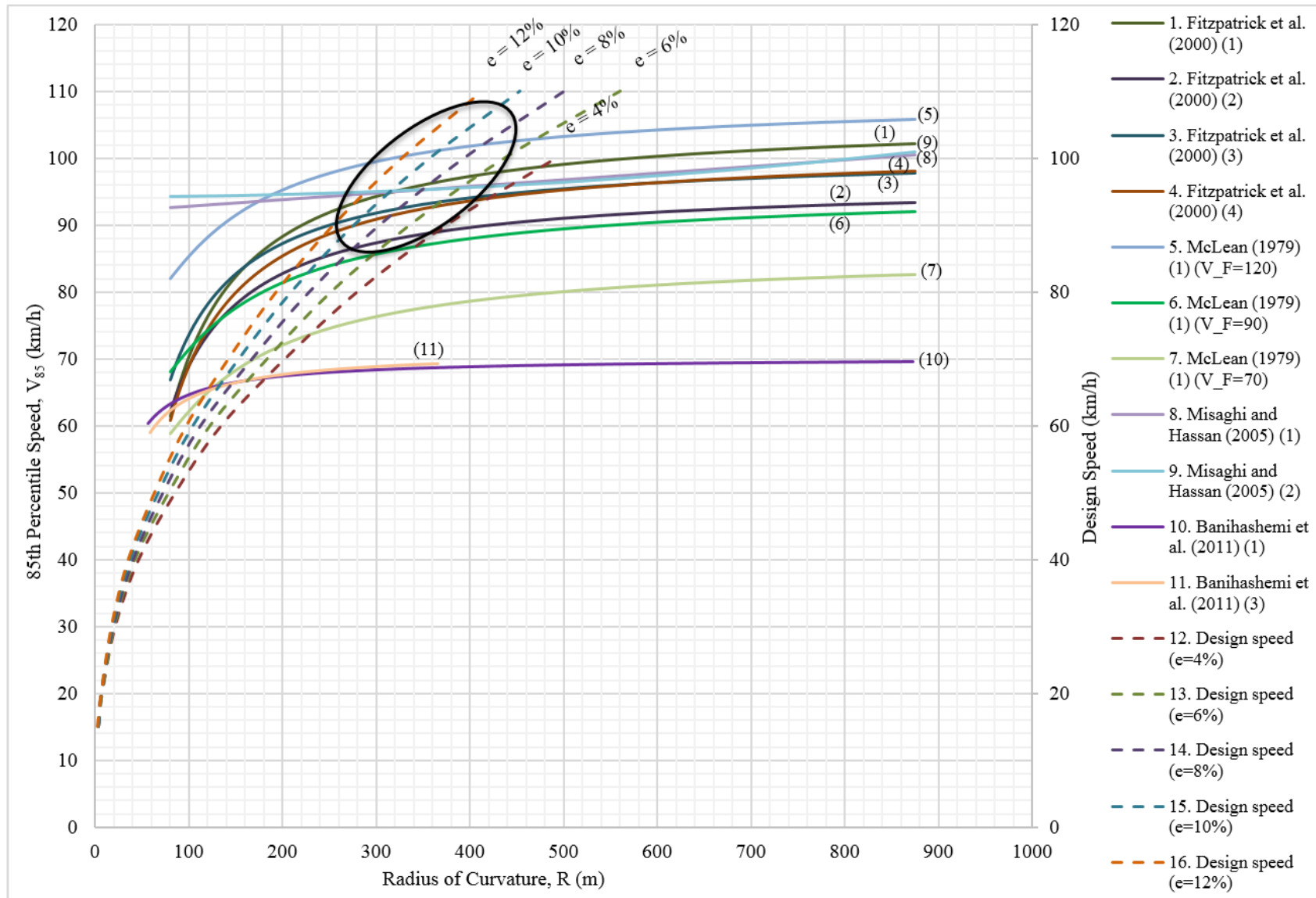
Figure 13. Graph. Radius of curvature versus 85th-percentile speeds and design speeds.

Source: FHWA.
Note: 1 km/h = 0.621371 mph; 1 m = 3.28 ft; in the legend, single numbers that appear in parentheses after the publication year are the equation numbers used from that publication.

As shown in figure 13, there are several speed-inverse radius-of-curvature relationships. The nonlinear portion of the 85th-percentile speed lines show a steep incline when the radius of curvature is small but begin to level as the radius increases. It appears that horizontal curve radii less than 985 ft (300 m) have the greatest influence on vehicle operating speeds. Equations 1 and 3 by Banihashemi et al. (2011) were created for low-speed rural two-lane highways, while the remaining equations were for high-speed, two-lane rural highways.

In figure 13, the area approximately within the black oval represents the range in which design speeds and operating speeds are similar. For very sharp curves, the geometry of the roadway tends to influence the operating speed of vehicles. Depending on the superelevation of the road, horizontal curvature tends to have little effect on operating speeds when the radius of curvature is approximately 1,480 ft (450 m) or larger. Readers interested in the association between the radius of curve and the expected number of crashes on two-lane rural highways should refer to the AASHTO *Highway Safety Manual* (HSM). (AASHTO 2010)

Relationship Between Degree of Curvature and Speeds

Equation 1 by Krammes et al. (1995) and equations 2, 4, 6, and 8 by Lamm and Choueiri (1987) were plotted to show the relationship between the degree of curvature and 85th-percentile operating speeds in figure 14. Lane widths were considered in three of the equations: the Lamm and Choueiri (1987) (equation 4) model is applied for 10-ft (3.0-m) lane widths, the model by Lamm and Choueiri (1987) (equation 6) is applied for 11-ft (3.4-m) lane widths, and the model by Lamm and Choueiri (1987) (equation 8) is applied for 12-ft (3.7-m) lane widths.
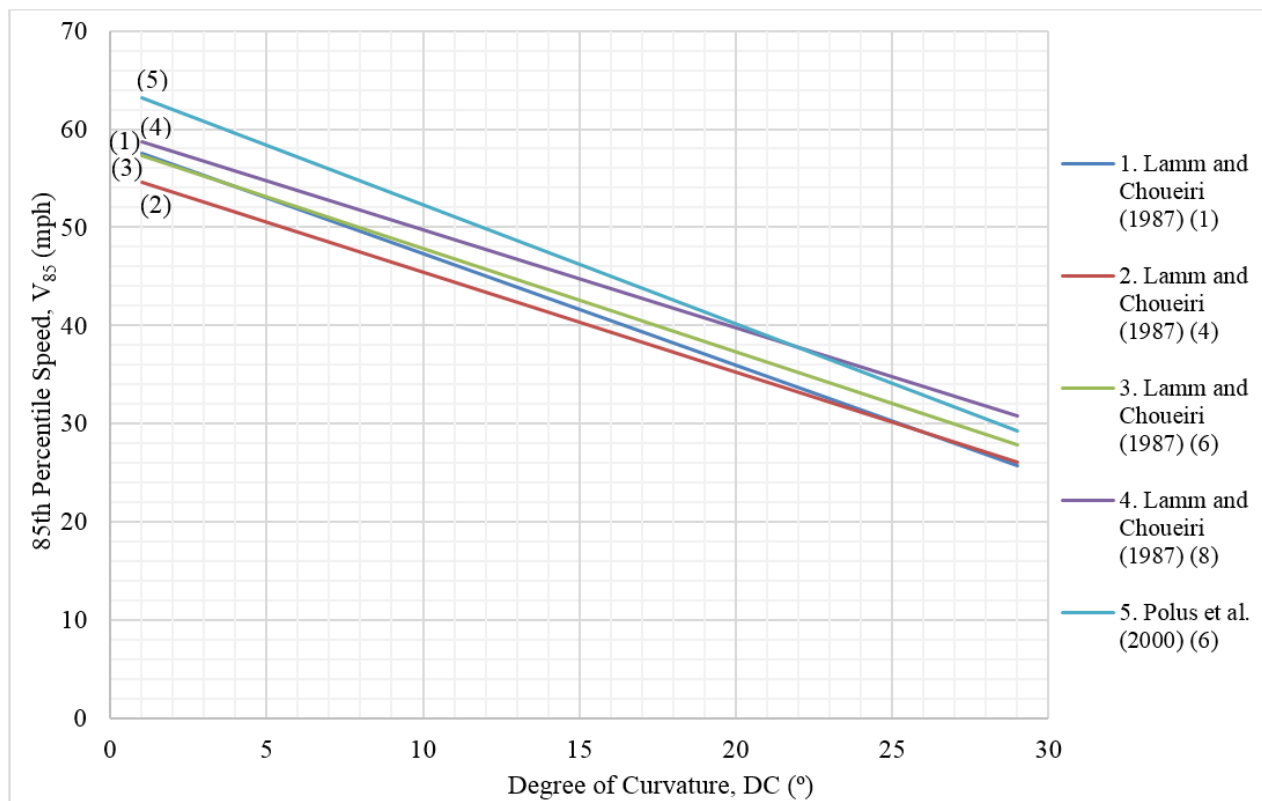
Source: FHWA.
Note: 1 mph = 1.60934 km/h; in the legend, single numbers that appear in parentheses after the publication year are the equation numbers used from that publication.

Figure 14. Graph. Degree of curvature versus 85th-percentile speeds.

Figure 14 shows that as the degree of curvature increases, the 85th-percentile operating speed decreases. As the lane width increases, the 85th-percentile operating speed correspondingly increases.

Relationship Between Curves, Tangent Length, and Speed

Polus et al. (2000) estimated statistical models of 85th-percentile operating speeds on tangent segments of two-lane rural highways by considering the horizontal curve radii of distal and proximal curves (previous and following curves). For long tangent lengths that exceed 492 ft (150 m), the geometric measure of the tangent section and adjacent curves is represented in the model in figure 15, which is then used in one of the speed prediction equations shown in figure 17.

$$GM_L = \left[TL \times (R_1 \times R_2)^{1/2}\right]/100 \; for \; TL \geq t$$

Figure 15. Equation. Geometric measure of tangent section and attached curves for long tangent length.

Where:

GM_L = geometric measure of tangent section and attached curves for long tangent length (ft^2 (m^2)).

R_1, R_2 = previous and following curve radii (ft (m)).

TL = tangent length (ft (m)).

t = selected threshold for tangent length (ft (m)).

For short tangent lengths, defined as less than 492 ft (150 m), the geometric measure of the tangent section and adjacent curves is represented in the equation in figure 16, which can then be used in the speed prediction model shown in figure 18.

$$GM_s = (R_1 + R_2)/2 \; for \; TL < t$$

Figure 16. Equation. Geometric measure for short tangent lengths.

Where GM_s is the geometric measure for short tangent lengths (ft (m)).

The 85th-percentile speed prediction equations developed by Polus et al. (2000) are shown in table 33 in appendix A. Speed prediction equations developed by Polus et al. (2000) (equations 1–6 in their report) which contain the variable GM_L, are shown in figure 17. As the values for GM_L increase, the 85th-percentile speeds increase. There are limitations on each equation for values of GM_L, tangent length, and radius of curvature that are noted in appendix A. The tangent length is kept constant in all equations. The two radii used are not specified; however, the product of the two radii in the equation for GM_L is increasing as the 85th-percentile speeds are increasing.
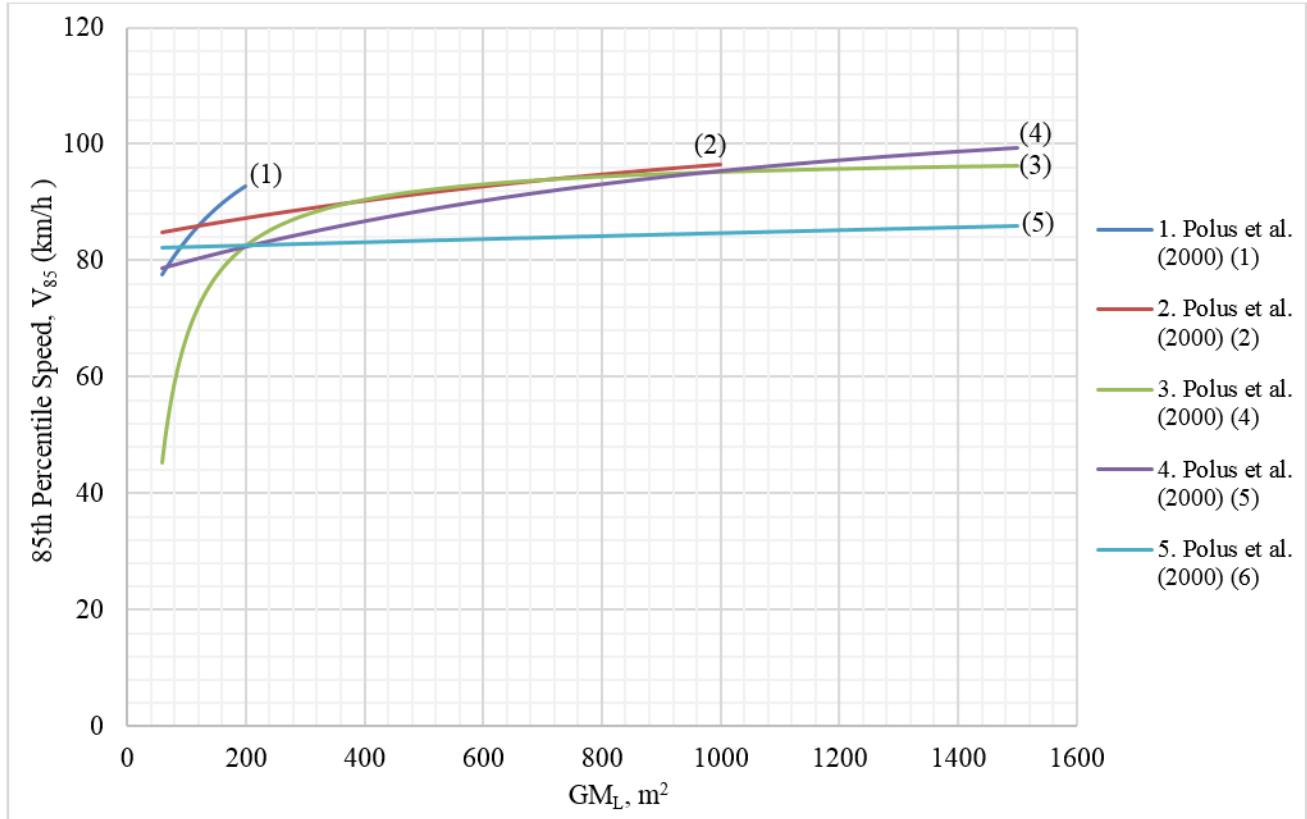
Source: FHWA.

Note: 1 km/h = 0.621371 mph; 1 m² = 10.7639 ft²; in the legend, single numbers that appear in parentheses after the publication year are the equation numbers used from that publication.

Figure 17. Graph. Geometric measure for long tangent lengths and attached curves versus 85th-percentile speeds.

Similar to the equations containing the variable GM_L, the equation by Polus et al. (2000) (equation 3) is illustrated in figure 18. While this equation is used for short tangent lengths (less than 492 ft (150 m)), as GM_s increases, so does the 85th-percentile operating speed. A larger GM_s indicates that the sum of both radii is larger. The tangent length is not factored into the equation for GM_s due to its small value.

Source: FHWA.
Note: 1 km/h = 0.621371 mph; 1 m = 3.28 ft; in the legend, the single number that appears in parentheses after the publication year is the equation number used from that publication.

Figure 18. Graph. Geometric measure of short tangent section versus 85th-percentile speeds.

The following statistical model from Polus et al. (2000), as shown in figure 19, was used to generate the speed–tangent relationship:

$$SP = 105.00 - 21.30/e^{(0.00092 \times GM_L)}$$

Figure 19. Equation. Model for the speed–tangent relationship. (Polus et al. 2000)

Where *SP* equals the 85th-percentile speed (km/h) (1 km/h = 0.621371 mph).

This model is illustrated using a superelevation of 12 percent and a minimum radius of curvature for R_1 and R_2 for each designated design speed shown in figure 20, which is based on information in the Green Book. (AASHTO 2011) Additionally, each minimum radius of curvature for R_1 and R_2 was multiplied by 1.5 to show the effects of choosing larger-than-minimum radii on the speed–tangent length relationship.
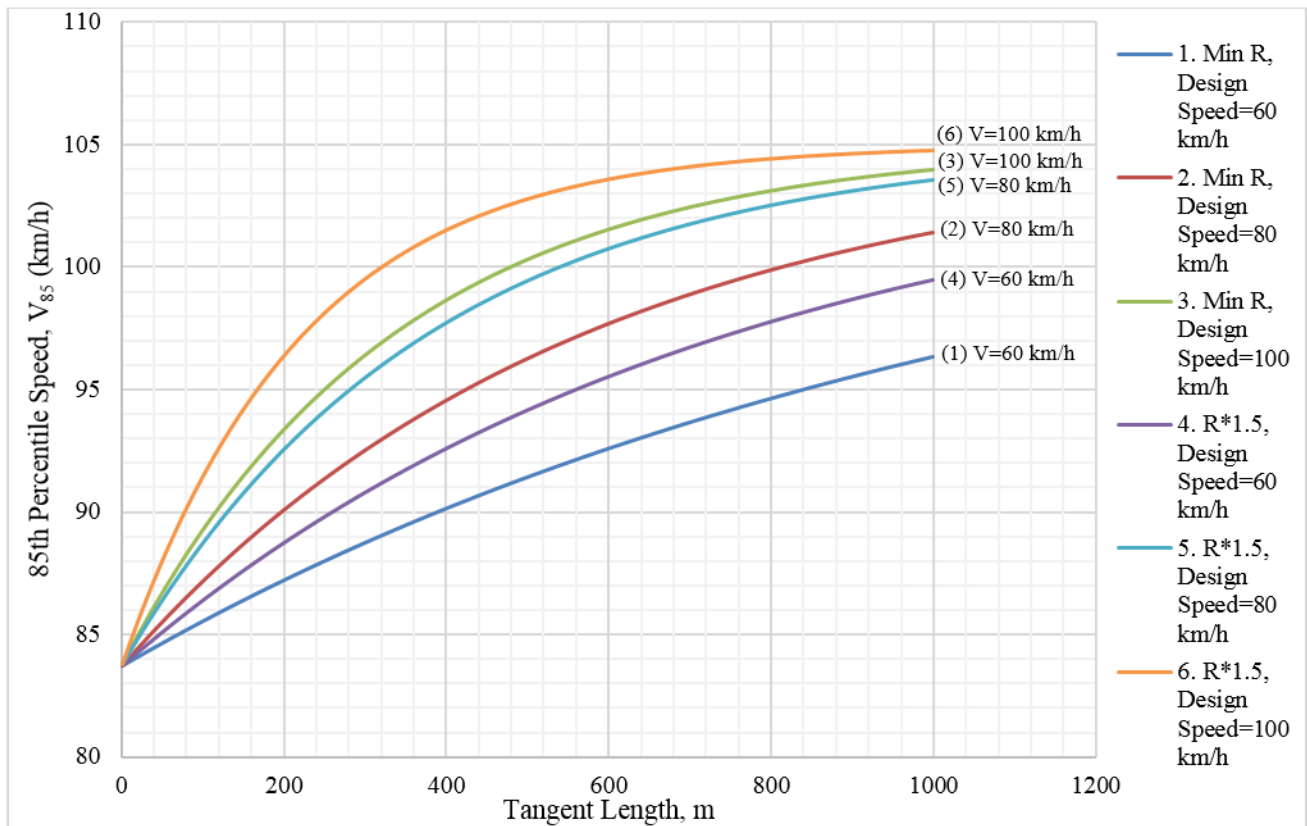
Source: FHWA.
Note: 1 km/h = 0.621371 mph; 1 m = 3.28 ft.

Figure 20. Graph. Tangent lengths versus 85th-percentile speeds for *e* = 12 percent.

As shown in figure 20, as tangent lengths between two horizontal curve radii increase, the 85th-percentile speeds increase. The shape of the curves shows a higher rate of change for tangent lengths and 85th-percentile speeds up to a certain tangent length, after which the influence of the tangent length on operating speed diminishes. Tangent lengths tend to significantly affect operating speeds until approximately 1,310 ft (400 m), at which point the tangent length does not have a substantial effect on speeds. When the designated design speed is higher (e.g., 62.14 mph (100 km/h)) the curves in figure 20 are sharper than at lower design speeds. Additionally, the 85th-percentile operating speeds are larger for curves with larger radii than curves with smaller radii.

VERTICAL ALIGNMENT AND SPEED RELATIONSHIP

There is a relationship between the vertical alignment of a roadway and design and operating speeds. The following section describes the vertical alignment design process and shows how the designated design speed is associated with the vertical alignment design elements. The section also illustrates how vertical alignment design decisions are associated with operating speeds. This information can be used to identify vertical alignment dimensions that produce operating speeds consistent with the designated design speed and posted speed limit along the roadway.

Vertical Curve Design

Similar to horizontal curves, sight distance on crest vertical curves must also be considered in the geometric design of highways and streets. Sight distance for vertical curves pertains to the driver's ability to see the road ahead when the vertical features of the roadway change. (AASHTO 2011) The minimum length of the vertical curve considers the algebraic difference in grades, SSD, height of the driver's eye above the roadway surface, and the height of an object above roadway surface. Crest vertical curve design is indirectly related to the designated design speed through SSD criteria.

There are two different models that may be used to determine the minimum length of crest vertical curves, depending on the relationship between the SSD and vertical curve length. The model used to determine the crest vertical curve length when the sight distance is less than the length, according to the Green Book, is as shown in figure 21.

$$L = \frac{AS^2}{100\left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2}$$

Figure 21. Equation. Crest vertical curve length when sight distance is less than the vertical curve length.

Where:
  L = length of vertical curve (ft (m)).
  A = algebraic difference in grades (percent).
  S = sight distance (ft (m)).
  h_1 = height of eye above roadway surface (ft (m)).
  h_2 = height of object above roadway surface (ft (m)).

When the sight distance is greater than the vertical curve length, the model is as shown in figure 22.

$$L = 2S - \frac{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}{A}$$

Figure 22. Equation. Crest vertical curve length when sight distance is greater than the vertical curve length.

The length of sag vertical curves is affected by headlight sight distance. Sag vertical curves consider the algebraic difference in grades and headlamp beam distance. According to the Green Book, the headlamp beam distance is "the distance between the vehicle and point where the 1-degree upward angle of the light beam intersects the surface of the roadway." (AASHTO 2011) The length of sag vertical curve is indirectly related to the designated design speed of the roadway via the SSD.

Figure 23 through figure 26, which are from the Green Book, illustrate the computations needed to determine the length of a sag vertical curve for various stated conditions. (AASHTO 2011)

When the headlamp beam distance is less than the length of the sag vertical curve, the equation from either figure 23 or figure 24 is used.

$$L = \frac{AS^2}{200[2.0 + S(\tan 1°)]}$$

Figure 23. Equation. Length of sag vertical curve when headlamp beam distance is less than the length. (AASHTO 2011)

$$L = \frac{AS^2}{400 + 3.5S}$$

Figure 24. Equation. Length of sag vertical curve when headlamp beam distance is less than the length-reduced equation. (AASHTO 2011)

When the headlamp beam distance is greater than the length of the sag vertical curve, the equation from either figure 25 or figure 26 is used.

$$L = 2S - \frac{200[2.0 + S(\tan 1°)]}{A}$$

Figure 25. Equation. Length of sag vertical curve when headlamp beam distance is greater than the length. (AASHTO 2011)

$$L = 2S - \frac{400 + 3.5S}{A}$$

Figure 26. Equation. Length of sag vertical curve when headlamp beam distance is greater than the length reduced equation. (AASHTO 2011)

Relationship Between Rate of Vertical Curvature and Operating Speeds

When equations 5 and 6 by Fitzpatrick et al. (2000a) were plotted to illustrate the relationship between rate of vertical curvature and 85th-percentile operating speeds, as shown in figure 27, the shape resembles the relationship between horizontal radius of curvature and 85th-percentile operating speed. The graphed equations for 85th-percentile speed were compared to the design speed based on the rate of vertical curvature for SSD from table 3-34 in the AASHTO Green Book. (AASHTO 2011) The design speeds are shown on the right (secondary) vertical axis of figure 27.
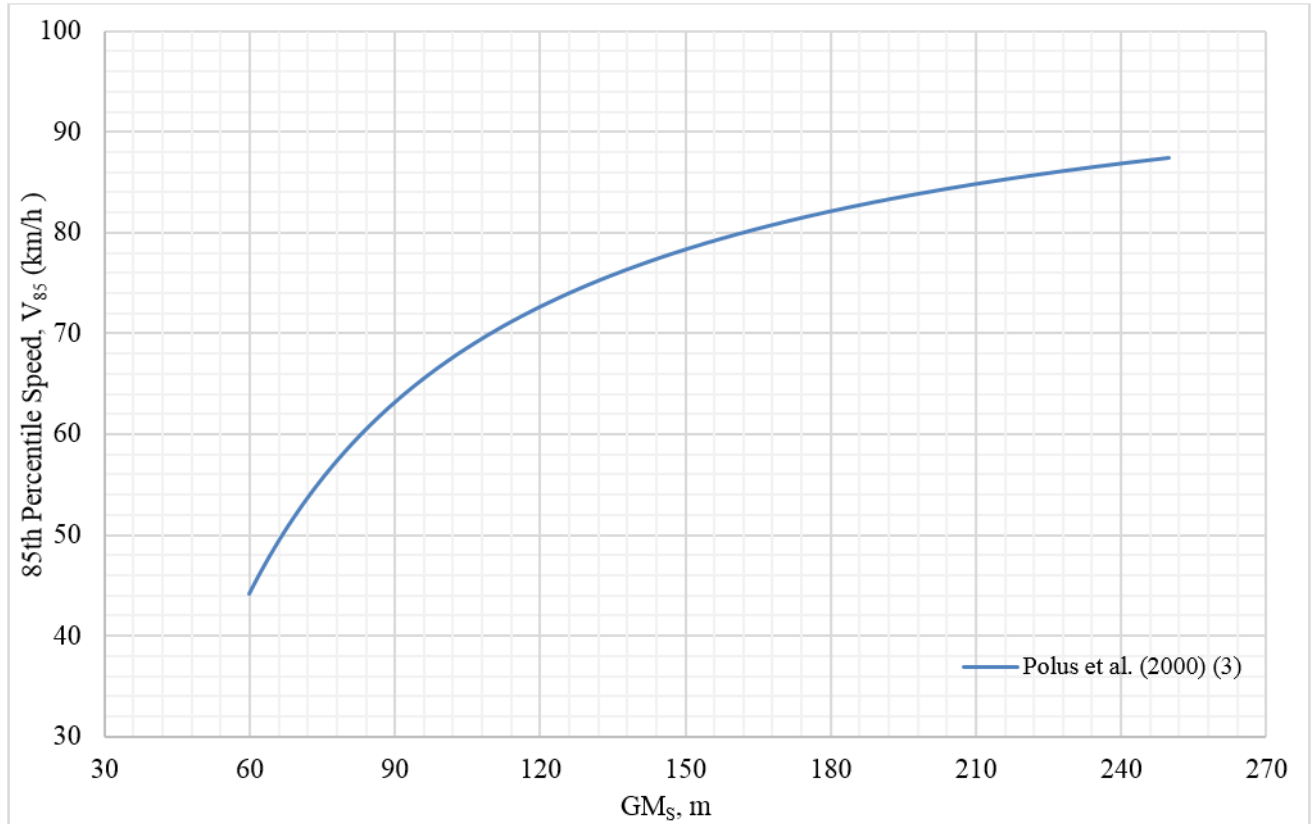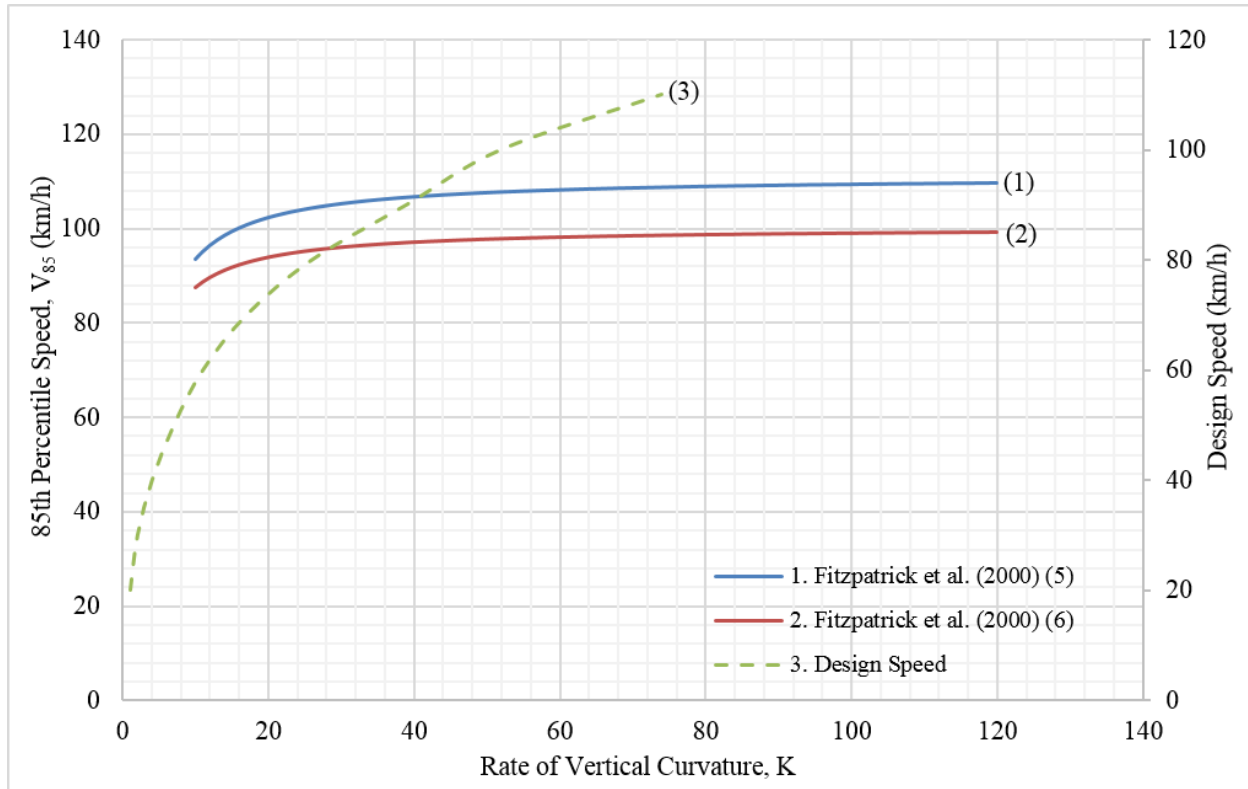
Source: FHWA.
Note: 1 km/h = 0.621371 mph; in the legend, single numbers that appear in parentheses after the publication year are the equation numbers used from that publication.

Figure 27. Graph. Rate of vertical curvature versus 85th-percentile speeds and design speeds.

Equation 5 by Fitzpatrick et al. (2000a) was used to plot the speed-vertical curve relationship in figure 27 for vertical curves with limited SSD on horizontal tangents, while equation 6 by Fitzpatrick et al. (2000a) was used for sag vertical curves on horizontal tangents with limited sight distance. (Fitzpatrick et al. 2000a) Both show a sharp increase in 85th-percentile operating speeds when the rate of vertical curvature is between approximately 10 and 29, and then the slope of the graphic increases slowly as the rate of vertical curvature increases. Additionally, when the recommended minimum rates of vertical curvature are used, there is a greater influence on the 85th-percentile speeds for both SSD and passing sight distance.

CROSS SECTION AND SPEED RELATIONSHIP

There is no well-documented relationship between roadway cross-section elements and operating speeds on rural two-lane highways. However, the designated design speed of the roadway is associated with several cross-section elements on two-lane rural highways. Cross-section elements can include, but are not limited to, shoulder widths, lane widths, number of lanes, and roadside features. The following section describes the design process for cross-section features and discusses operating speed models that show predicted operating speeds based on the various designed cross-section elements.

Cross-Section Design

Cross-section elements that are related to the designated design speed of a roadway include lane width, number of lanes, shoulder widths, and roadside features. According to the Green Book, the roadway is defined as "a portion of a highway, including shoulders, for vehicular use." (AASHTO 2011) Driving behavior, such as the selection of speeds, is influenced by the cross-sectional elements of a roadway. This section of the report describes how the designated design speed is related to cross-section dimensions, particularly on two-lane rural highways.

The roadway width and the number of lanes are dependent on the designated design speed and design volumes. (AASHTO 2011) The number of lanes is also influenced by the target level of service and capacity requirements. (AASHTO 2011) The roadway width may also vary if accommodating the presence of bicyclists. The Green Book provides guidance for the minimum traveled-way widths for rural arterials that are determined through the designated design speed and design volume. It also states minimum widths of usable shoulders based on design volumes. (AASHTO 2011)

The Green Book offers general guidance for lane width dimensions, which range from 9 to 12 ft (2.7 to 3.7 m), based on the roadway type and traffic volume. (AASHTO 2011) On high-speed, high-volume roadways, 12-ft (3.7-m) lanes are recommended. Lane widths of 10 ft (3.0 m) can be used on low-speed roadways, while a 9-ft (2.7-m) width may be used on low-speed, low-volume roadways. (AASHTO 2011) Table 13 and table 14 show the Green Book recommended minimum traveled-way widths for rural arterials, based on the designated design speed and design volume. (AASHTO 2011) As shown in table 13 and table 14, lane widths of 11 or 12 ft (3.4 or 3.7 m) are recommended, depending on the designated design speed and design volume.

Table 13. Minimum width of traveled way for rural arterials (AASHTO 2011, table 7-3).

Design Speed (mph (km/h))	Design Volume (vehicles/d) Under 400	Design Volume (vehicles/d) 400 to 1,500	Design Volume (vehicles/d) 1,500 to 2,000	Design Volume (vehicles/d) Over 2,000
40 (64.4)	22	22	22	24
45 (72.4)	22	22	22	24
50 (80.5)	22	22	24	24
55 (88.5)	22	22	24	24
60 (96.6)	24	24	24	24
65 (104.6)	24	24	24	24
70 (112.7)	24	24	24	24
75 (120.7)	24	24	24	24

Table 14. Minimum width of usable shoulder for rural arterials (AASHTO 2011, table 7-3).

Design Speed (mph (km/h))	Design Volume (vehicles/d) Under 400	Design Volume (vehicles/d) 400 to 1,500	Design Volume (vehicles/d) 1,500 to 2,000	Design Volume (vehicles/d) Over 2,000
All speeds	4	6	6	8

According to the Green Book, "a shoulder is the portion of the roadway contiguous with the traveled way that accommodates stopped vehicles, emergency use, and lateral support of subbase, base, and surface course." (AASHTO 2011) Shoulders can be paved or unpaved. Shoulder width design guidance varies depending on the functional class and planned use of the shoulder. According to the Green Book, shoulder widths are typically 12 ft (3.7 m) for higher speed roads with high traffic volumes and a significant truck proportion among the traffic, typically referring to freeways, while 6-ft (1.8-m) shoulders are more common on low-volume roads. Table 14 shows the recommended minimum width of the usable shoulder for all design speeds based on design volumes. Shoulder widths of 4 to 8 ft (1.2 to 2.4 m) are recommended on rural arterials.

The roadside is the area beyond the shoulders and is considered part of the cross section. AASHTO's *Roadside Design Guide* offers guidance for the design of roadside features (specifically, clear-zone distances) based on the designated design speed and design average daily traffic. (AASHTO 2011) Higher design speeds require larger clear zones. Additionally, the clear-zone requirements also increase as the design average daily traffic increases. Typically, steeper foreslopes and backslopes are associated with wider clear-zone recommendations. Readers interested in the association between cross-section elements and the expected number of crashes on two-lane rural highways should refer to the AASHTO HSM. (AASHTO 2010)

CHAPTER 4. SELF-ENFORCING ROAD CONCEPTS

The self-enforcing roadway concepts may be applied to planned or existing roadways that are programmed for reconstruction. As noted previously, a self-enforcing road is a roadway that is planned and designed to encourage drivers to select operating speeds in harmony with the posted speed limit. The objective of self-enforcing roads is to produce speed compliance. Self-enforcing roads aim to change driver behavior by using geometric elements resulting in operating speeds commensurate with the intended roadway purpose, including the adjacent land use.

According to Fitzpatrick et al. (2003), "a design process is desired that can produce roadway designs that result in a more harmonious relationship between the desired operating speed, the actual operating speed, and the posted speed limit." While it is thought that achieving speed harmony in geometric design affects the safety of a road, the actual effects are unknown. (Porter et al. 2012) However, when speed harmony exists, the road designs "look and feel" like the intended purpose and can be described as more self-enforcing, or self-explaining. (Fitzpatrick et al. 2003) However, when the operating speeds of a roadway are inconsistent with the design speed in which the roadway features were developed, speed discord results. Speed discord has been defined as a roadway design that produces operating speeds that are higher than the posted speed limit. Donnell et al. (2009) found that speed discord on two-lane rural highways often resulted from the use of above-minimum values of geometric design criteria. In such cases, the 85th-percentile operating speed along a roadway segment often exceeded the posted speed limit and the designated design speed.

Several geometric design procedures have been developed to overcome some of the limitations associated with the design speed concept. Application of minimum (or limiting) value of geometric design criteria in combination with larger-than-minimum (or less-than-limiting) values of criteria may produce design inconsistencies. The concepts described below may be used to design roadways that produce operating speeds consistent with the desired operating speeds of the roadway. The following section describes guidelines for the following six methods that can be used to design self-enforcing roadways:

1. Applying the speed feedback loop process.
2. Using the inferred design speed approach.
3. Applying operating speed models.
4. Utilizing existing geometric design criteria.
5. Using a combination of signs and pavement markings.
6. Setting rational speed limits.

METHOD 1—SPEED FEEDBACK LOOP

One geometric design procedure that reflects the self-enforcing road concept is incorporating a speed feedback loop into the design process. The speed feedback loop process requires checking for consistency among design elements and anticipated operating speeds to determine an appropriate posted speed limit. Use of a speed feedback loop during the design process involves predicting speeds during preliminary engineering and comparing the expected operating speed to

the geometric design speed. The objective is to produce a design where the expected operating speed is equal to, or nominally lower than, the designated design speed.

The speed feedback loop process described later in this section is similar to that used in the *Geometric Design Guide for Canadian Roads*; however, the book refers to the process as the "design domain" concept. (Transportation Association of Canada (TAC) 1999) The design domain concept is a method used to select an appropriate design speed based on a process similar to that of the speed feedback loop. Separate processes are recommended for different road types (e.g., there are different processes for two-lane rural roads, divided roadways, and urban roadways). For two-lane rural local, collector, or arterial roads, the design domain concept process is as follows:

- Select a nominal (trial) design speed.

- Select the design parameters for vertical and horizontal alignment and other highway geometric elements.

- Develop a trial alignment.

- Estimate the 85th-percentile speeds on the trial alignment (refer to section 1.4 in the *Geometric Design Guide for Canadian Roads*).

- Check consistency; does the estimated speed match the design speed?

- If the estimated speed matches the design speed, finalize the design.

- If the estimated speeds do not match the design speed, can the alignment be modified? If so, develop a new trial alignment.

- If the alignment cannot be modified, select another nominal (trial) design speed and repeat the process. (TAC 1999)

While there are multiple ways to determine the design speed described above, the guide recommends setting the design speed equal to the existing 85th-percentile speed. (TAC 1999) This operating speed approach for the design of two-lane highways is shown in figure 28.

Select Design Speed

Select Design Parameters for Highway Geometric Elements

Trial Alignment

Estimate 85th-Percentile Speeds

Check Consistency. Does Estimated Speed Match Design Speed?

No

Can the Alignment Be Modified?

No

Yes

Yes

Satisfactory Alignment Final Design
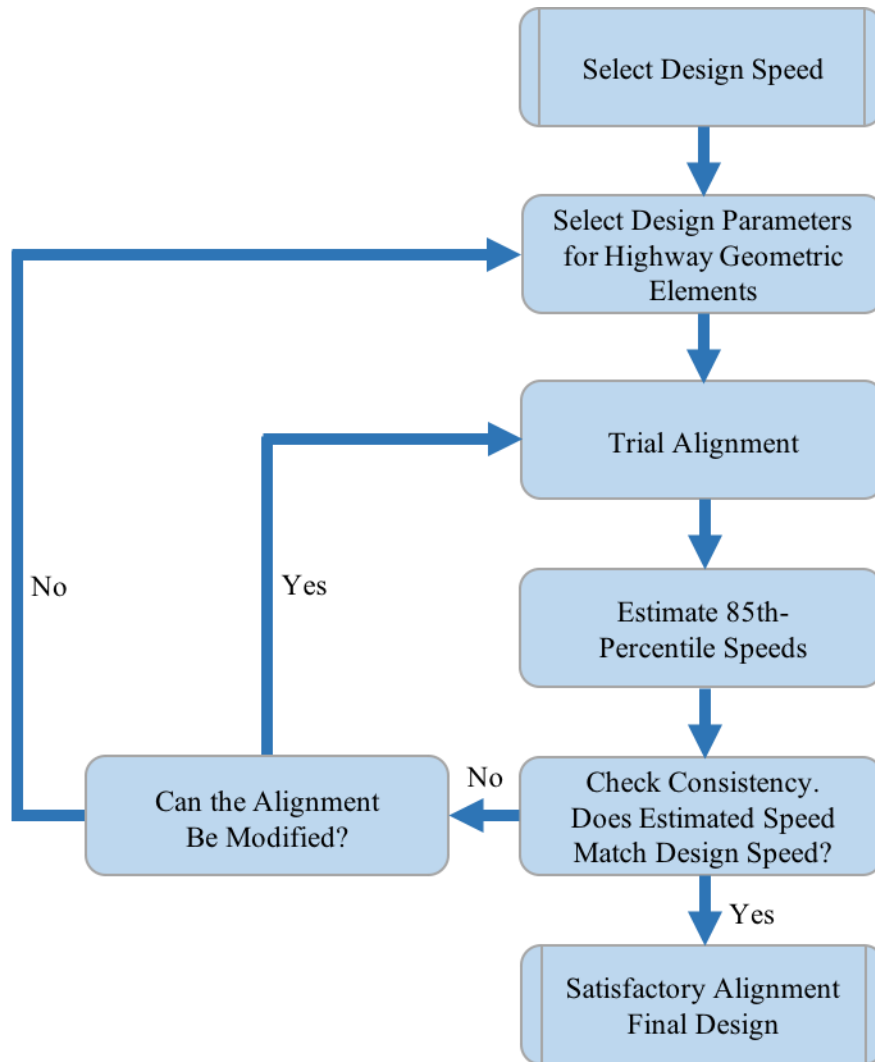
©Transportation Association of Canada. Reproduced with the express written authority of Transportation Association of Canada (TAC). Excerpted from the TAC publication *Geometric Design Guide for Canadian Roads* (1999).

Figure 28. Flowchart. Operating speed approach for design of two-lane, two-way roadways. (TAC 1999, figure 1.2.3.1)

Similar to the design approach utilized in Canada and presented in figure 28, a speed feedback loop may be integrated into the existing AASHTO Green Book geometric design process. (AASHTO 2011) A proposed framework for a speed feedback loop process within the existing AASHTO Green Book geometric design framework is shown in figure 29.
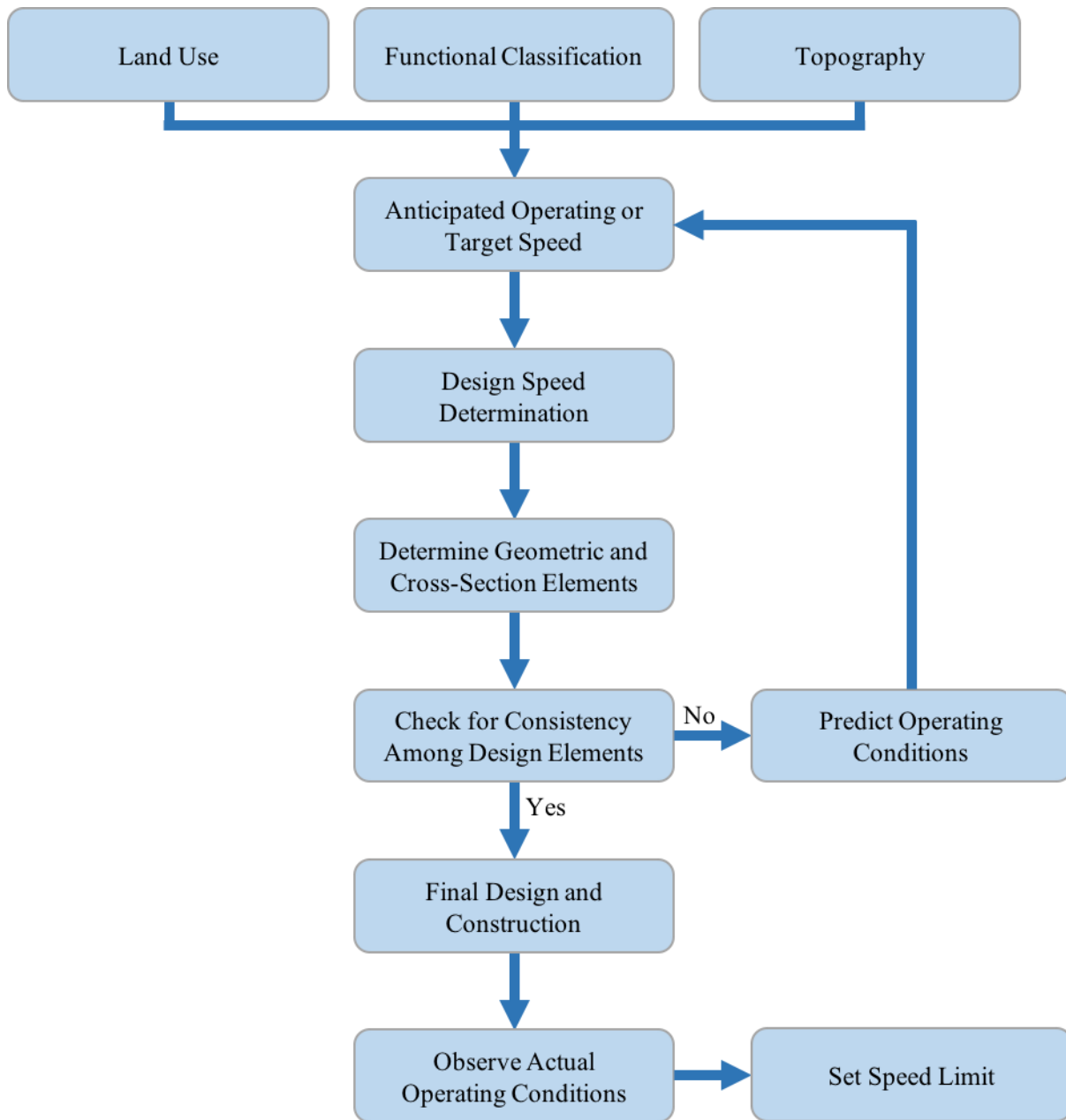
©ITE. Reproduced with the express written authority of ITE CDROM Proceedings of the ITE 2002 Annual Meeting and Exhibit.

Figure 29. Flowchart. Proposed framework to improve design speed concept. (Adapted from Donnell et al. 2002, figure 4)

The primary modification to the existing AASHTO geometric design process is the "check for consistency among design elements" step in figure 29. A detailed description of how this step can be integrated into the existing geometric design process is as follows:

Step 1—Consider the Land Use, Topography, and Functional Class

The first step in the speed feedback loop process is to consider and record the existing and intended land use of the surrounding area, the topography, and the intended functional class of the roadway. The considerations can include rural, suburban, or urban environments and level, rolling, or mountainous terrain. Electronic databases such as Google® Maps™ or field visits can aid in determining land use, topography, and functional class. Land use maps may also be used to identify intended land uses.

Step 2—Determine a Target or Anticipated Operating Speed

The land use, topography, and functional class, considered in step 1, are then used to determine the target speed or anticipated operating speed. For example, lower operating speeds may be anticipated in urban environments, while higher speeds might be expected on rural highways.

Step 3—Determine the Design Speed of the Roadway

The third step of the speed feedback loop process is to select an appropriate designated design speed, which will then be used to determine the geometric and cross-section elements of the roadway. This is accomplished using the target or anticipated operating speed determined in step 2. For the self-enforcing road concept, the design speed can be replaced by the target speed because the target speed thus becomes the posted speed limit. (ITE 2010) The designated design speed is chosen based on the target speed such that it is consistent with the target or anticipated operating speed. The Green Book recommends design speeds for roadways based on different functional classification and terrain type. (AASHTO 2011) However, the target speed becomes the primary control used in determining the geometric design values for roadway features. (ITE 2010) Refer to chapter 5 of the Green Book for Local Roads and Streets, chapter 6 for Collector Roads and Streets, and chapter 7 for Rural and Urban Arterials.

Step 4—Acquire Roadway Geometric Information

Once the designated design speed is determined (in step 3), the Green Book design criteria can be used to establish the geometric features of the roadway. (AASHTO 2011) The radius of curvature, SSD, length of vertical curvature, and HSO are directly related to the designated design speed. The lane and shoulder width on two-lane rural highways can be determined based on the roadway classification, the design traffic volume, typical cross-sections, and the available cross-section width.

Step 5—Check for Consistency Between Design Elements and Anticipated Operating Speeds

Once all geometric and cross-section elements have been designed, the next step of the speed feedback loop process involves checking for consistency between design elements. Consistency between design elements may result in operating speed uniformity along highway corridors. To assess geometric design consistency, 85th-percentile operating speed models may be used, such as those described briefly in this section; however, a more detailed discussion of operating speed models can be found in the method 3 discussion below. Chapter 4 also includes examples of

operating speed prediction models that may be used to check for consistency between design elements.

Site-specific features are used as input variables in the operating speed model and to estimate 85th-percentile operating speed. This output is compared for successive roadway design elements and compared to design consistency performance criteria. Design consistency criteria from the IHSDM can be used to assess the expected results of the initial geometrics. The criteria are as follows: "Good" is associated with speed differences less than 6 mph; speed differences greater than 6 mph (9.7 km/h) and less than 12 mph (19.3 km/h) are considered "fair"; and speed differences greater than 12 mph (19.3 km/h) are considered "poor." Example criteria include the difference in expected 85th-percentile operating speed and the geometric design speed for each feature along the roadway, or the difference in the 85th-percentile operating speed between successive design features. Large magnitude differences in design consistency metrics indicate that the design features are not producing operating speeds that are consistent between elements or with the designated design speed of the roadway.

The speed feedback loop process should be repeated. The geometric and cross-section elements should be redesigned to produce desired operating speeds. Once new operating speed values are calculated, the 85th-percentile speed prediction equations should be used again to check for consistency. If consistency has been achieved at a sufficient level, the project then advances into final design and construction.

Step 6—Set Posted Speed Limit

After the geometric elements have been designed and consistency has been achieved, the next step in the speed feedback loop design process is to determine an appropriate posted speed limit. The most common methods used to set a regulatory speed limit are as follows:

- **Engineering study.** This process involves measuring the operating speeds along a roadway. Vehicle operating speeds should be measured, based on a free-flow time headway criterion (typically 4 s or more), to calculate the 85th-percentile operating speed. The speed limit should be set at or within 5 mph (8.0 km/h) of the 85th-percentile operating speed. However, it can be adjusted based on crash history or special road conditions, such as access density, pedestrian and bicycle traffic, and parking if road conditions changed after the speed measurements. For additional information on engineering studies, refer to the MUTCD. (FHWA 2009)

- **Statutory speed limit (determined by law).** Statutory speed limits are legislated speed limits. In States that have adopted statutory speed limits, roadways of a certain category (e.g., business district, rural interstate highways, or alleys and residential streets) may have a regulatory limit that is mandated by law.

The speed limit should be set according to one of these two approaches.

METHOD 2—INFERRED DESIGN SPEED APPROACH

Another concept that can be utilized to design self-enforcing or self-explaining roadways is the inferred design speed approach. When creating a self-enforcing roadway, the inferred design

speed, operating speeds, and posted speed limit should be in harmony with one another. The inferred design speed approach is used to assess the relationship among these speed measures by plotting them graphically.

An inferred design speed is defined as "the maximum speed for which all critical design-speed-related criteria are met at a particular location" and applies only to design criteria that are based on the designated design speed. (Donnell et al. 2009) The designated design speed and inferred design speed will differ when using larger-than-minimum values (or lower-than-limiting values) of geometric design criteria. When applying this method, the inferred and designated design speeds are plotted on a two-dimensional plot (speed versus roadway length) to evaluate design consistency. This graphic can provide information about setting an appropriate regulatory speed limit that is related to the anticipated operating speeds of a roadway. Large differentials (defined as "poor" in method 1) between the inferred and designated design speeds will likely produce operating speeds that are higher than anticipated in the design process. Figure 30 displays an example of a speed profile based on the inferred design speed approach.
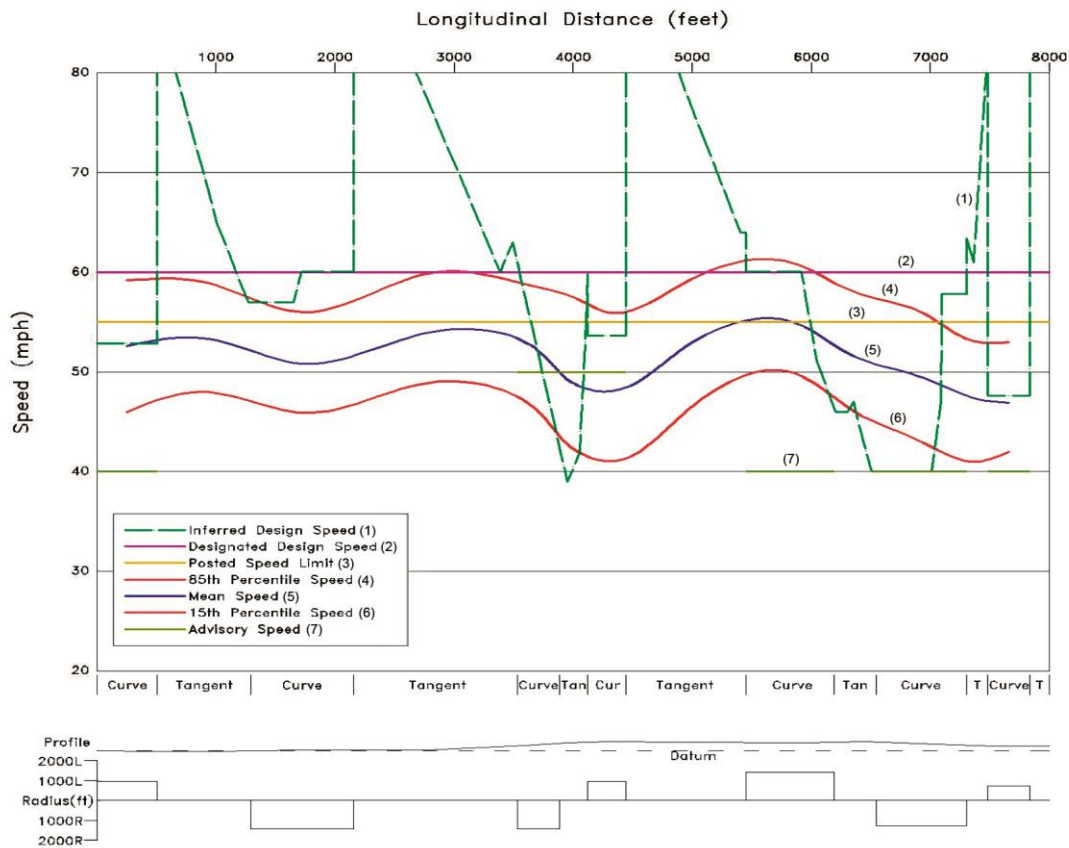
Source: FHWA.

Figure 30. Illustration. US Route 6 speed profile. (Donnell et al. 2009b)

The framework for the inferred design speed approach process is shown in figure 31. As illustrated, the inferred design speed approach can be used for existing or planned roadways. The information needed for either approach is shown. Example calculations to compute the inferred design speed based on existing or planned roadway features are shown in appendix B.

49
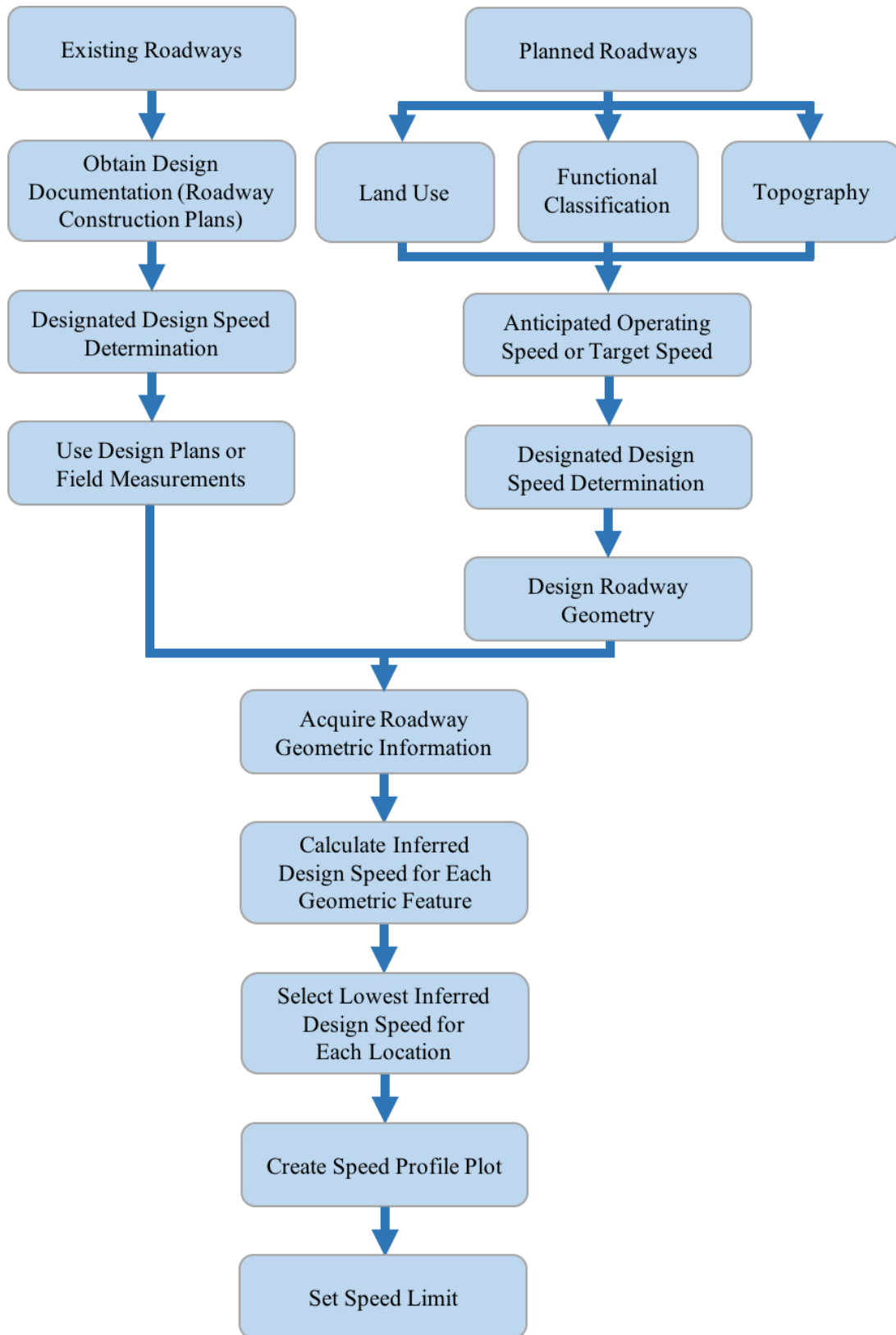
Source: FHWA.

Figure 31. Flowchart. Framework for inferred design speed approach.

Step 1—Determine the Designated Design Speed of the Roadway

The first step of the inferred design speed approach is to determine the designated design speed of the roadway, which is used to establish the horizontal alignment, vertical alignment, and cross-section elements of the planned roadway. For existing roadways, the designated design speed is typically found in the design documentation, usually located on the cover sheet of the roadway construction plans. For new roadways, the designated design speed needs to be selected. To choose a design speed, the land use, functional classification, and topography need to be identified and considered. Taking those characteristics into consideration, the target speed or anticipated operating speed can be determined, and the design speed can be set based on the target speed. This process is consistent with that found in the 2011 AASHTO Green Book.

Step 2—Acquire Roadway Geometric Information

The next step in the inferred design speed approach requires that all relevant roadway geometric information be used to determine the inferred design speeds. The geometric elements that need to be either identified for existing facilities or designed for planned facilities include the following:

- Radius or degree of curvature.
- SSD.
- Vertical curvature.
- Superelevation.
- HSO (along inside of horizontal curve).
- Lane width.
- Shoulder width.

For existing facilities, the dimensions of the geometric elements can be determined using the design plans or measured in the field. For planned facilities, this information can be found on the plan, profile, and cross-section sheets of the design plans.

Step 3—Calculate Inferred Design Speed for Geometric Features

Once all the dimensions of the geometric elements have been calculated and determined across the entire roadway segment, the inferred design speeds associated with each geometric element can be calculated. The inferred design speed is calculated differently for the various geometric features. Each method for calculating the inferred design speeds is described below.

- **Radius of curvature and superelevation.** There is a single inferred design speed for each combination of horizontal curvature and superelevation. The inferred design speed for the horizontal elements is based on the point mass equation shown in figure 32. The process is iterative to capture a friction factor that is as close to, but not exceeding, the maximum friction value for a given design speed.

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

Figure 32. Equation. Horizontal curve.

Where:

    V = speed (mph (km/h)).

    e = rate of superelevation (percent).

    f = side-friction demand factor.

- **Length of vertical curve.** The equations in figure 33, figure 34, and figure 35 are from the 2011 Green Book; they can be used to calculate the inferred design speed for crest vertical curves. (AASHTO 2011)

$$A = \ |G_2 - G_1|$$

Figure 33. Equation. Algebraic difference in grades. (AASHTO 2011)

Where G_1 and G_2 are grades (percent).

$$L = \frac{A(SSD^2)}{2158} \quad [for\ SSD < L]$$

Figure 34. Equation. Length of crest vertical curve when SSD is less than the length. (AASHTO 2011)

$$L = 2(SSD) - \frac{2158}{A} \quad [for\ SSD > L]$$

Figure 35. Equation. Length of crest vertical curve when SSD is greater than the length. (AASHTO 2011)

The equations in figure 36 and figure 37 are from the 2011 Green Book; they can be used to calculate the inferred design speed for sag vertical curves. (AASHTO 2011)

$$L = 2SSD - \frac{200(H + S \tan \beta)}{A} \quad [for\ SSD > L]$$

Figure 36. Equation. Length of sag vertical curve when SSD is greater than the length. (AASHTO 2011)

Where:

    β = incline angle of the headlight beam relative to the horizontal plane of the car.

    H = height of headlight above the roadway (ft (m)).

$$L = \frac{A(SSD^2)}{200(H + S \tan \beta)} \quad [for\ SSD < L]$$

Figure 37. Equation. Length of sag vertical curve when SSD is less than the length. (AASHTO 2011)

For the sag and crest vertical curve equations shown above, once the SSD has been determined, the inferred design speed can be determined using the Green Book and identifying which design speed is associated with the SSD.

- **SSD.** SSD can be determined through field measurements or using as-built plans. When the available SSD for a vertical curve is known, the 2011 Green Book equation shown in figure 38 can be used to calculate the inferred design speed, assuming near level vertical grades.

$$SSD = 1.47Vt + 1.075\frac{V^2}{a}$$

Figure 38. Equation. Available SSD. (AASHTO 2011)

Where:
SSD = available stopping sight distance (ft (m)).
V = inferred design speed (mph (km/h)).
t = perception–reaction time (2.5 s).
a = deceleration rate (11.2 ft/s^2 (km/s^2)).

When more than one inferred design speed is calculated for a location, the lower value controls the inferred design speed for that location. When a section of highway does not have an inferred design speed, such as a tangent section with no sight distance restrictions, the inferred design speed is infinity; however, a practical limit for such sections should be used. Examples from past research on two-lane rural highways have used 100 mph (160.9 km/h) as the maximum inferred design speed. (Donnell et al. 2009)

Step 4—Create a Speed Profile

After the inferred design speeds have been determined for all roadway sections, a speed profile plot can be created. An example speed profile is shown in figure 30. The designated design speed and inferred design speeds are plotted along a roadway section. On planned roadways, the inferred design speed, designated design speed, and predicted 85th-percentile operating speed should be plotted to assess how drive speed choice may relate to the design speed measures and the intended posted speed limit. For existing roadways, the speed profile plots may also include the 85th-percentile operating speed, posted speed limit, and other site-specific speed metrics (e.g., advisory speeds, mean speed).

Step 5—Set Speed Limit

Using the speed profile plot created in the previous step, an appropriate speed limit can be selected for the roadway. To create a self-enforcing roadway, the inferred design speed, actual or desired operating speeds, and posted speed limit should be within an acceptable range that should be determined early in the design process. The relationships between the speeds on the speed profile plot provide insights regarding an appropriate posted speed limit. An example of attaining speed harmony is when a posted speed limit is equal to the 85th-percentile operating speed, and both are equal to or lower than the designated or inferred design speed.

METHOD 3—DESIGN CONSISTENCY METHODS

The use of design consistency methods, such as operating speed models, is another method used to design self-enforcing roadways that involves setting speed limits based on a predicted

operating speed. These models aid in analyzing roadway design consistency by predicting operating speeds based on the geometry of the design. Design consistency can be evaluated manually using a series of equations, or the design values can be input into an automated computer operating speed model, such as the DCM of the FHWA's IHSDM.

While the equations produce anticipated operating speeds, the IHSDM is also able to estimate operating speeds, identify potential inconsistencies in speeds, and estimate crash frequency (using the Crash Prediction Module rather than the DCM), among other features. Using a design consistency method requires detailed geometric design data. These data are often not available in early project planning efforts or when geometric configurations are being planned. However, computer-aided files (or geometric design data) can be entered into the software. The IHSDM estimates 85th-percentile operating speeds, including acceleration and deceleration rates approaching and departing curves, and compares operating speeds on successive design elements. It also compares the designated design speed to the 85th-percentile operating speed. Speed differences less than 6 mph (9.7 km/h) are considered "good," while speed differences greater than 6 mph (9.7 km/h) are considered "fair" (if less than or equal to 12 mph (19.3 km/h)) or "poor" (if greater than 12 mph (19.3 km/h)).

The design consistency framework is shown in figure 39.
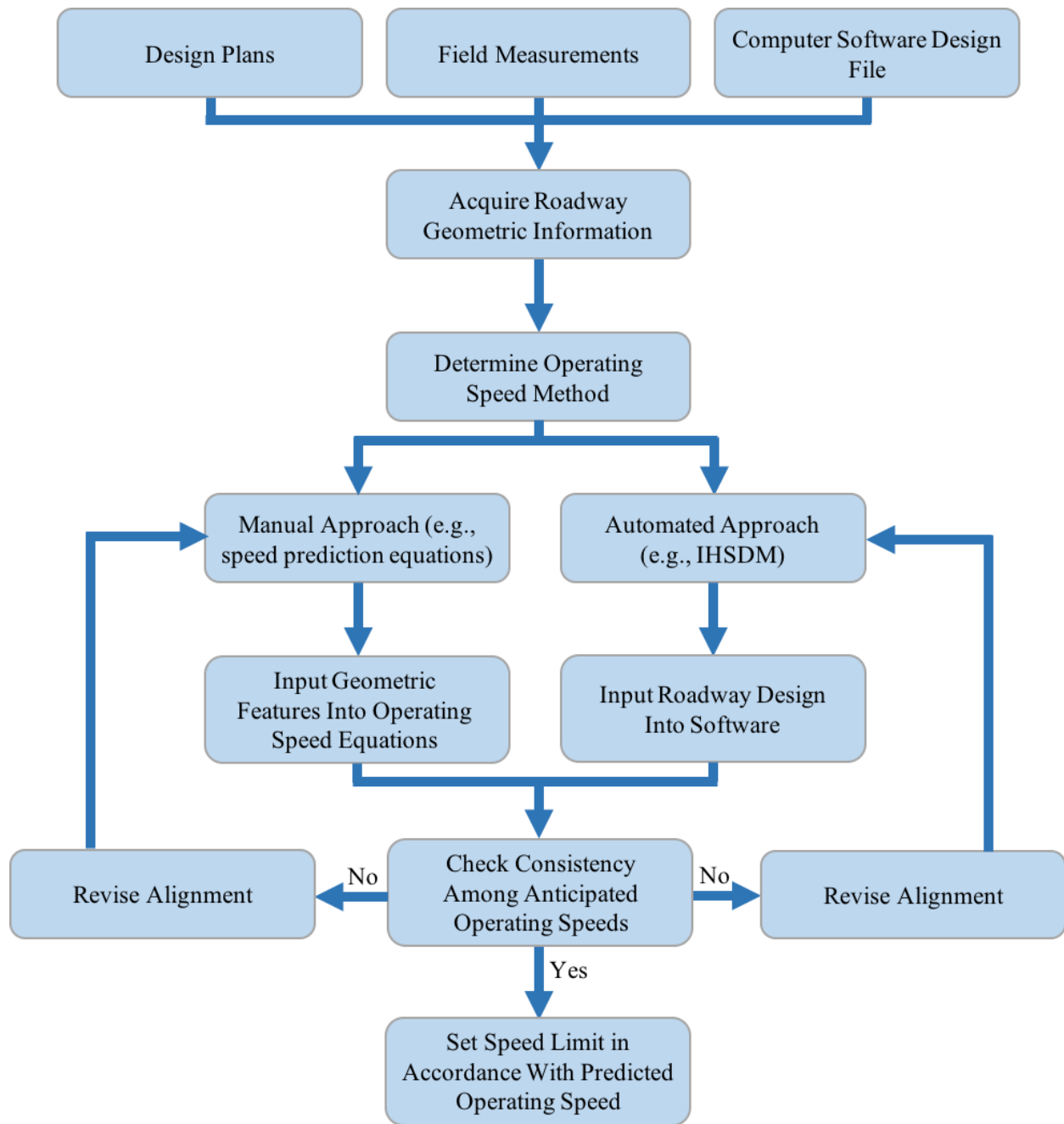
Source: FHWA.

Figure 39. Flowchart. Framework for using design consistency methods.

The process for designing a self-enforcing road using operating speed models is described below.

Step 1—Acquire Roadway Geometric Information

In order to apply the operating speed prediction models (equations) or to use the automated methods included in the ISHDM to design self-enforcing roads, specific roadway geometric design information is needed. The dimensions for the following geometric design features and other design components should be assembled when using the IHSDM DCM, if applicable:

- Radius of degree of curvature.
- Tangent lengths.
- Vertical curvature.
- Grade.
- Design speed.
- Desired speed.
- Posted speed limit.
- Roadside hazard rating.

Other geometric design feature dimensions that may be needed when using a design consistency method different from the IHSDM DCM are as follows:

- SSD.
- Superelevation.
- Lane width.
- Shoulder width.

The geometric information can be measured in the field for existing roadways or record drawings. Alternately, if available, design plans can be referenced for both existing and planned facilities.

Step 2—Determine Operating Speed Method to Use and Apply

After acquiring all roadway geometric information, the next step in the operating speed model approach to design self-enforcing roadways is to determine which operating speed method should be used. Manual approaches assess design consistency using equations developed by Fitzpatrick et al. (2000a, 2000b); automated design consistency approaches can apply computer software such as the IHSDM.

The 85th-percentile operating speeds can be approximated using the equations shown in table 15. Each equation shown has various alignment conditions for which it can be applied. Anticipated operating speeds should be calculated at all points along the roadway when the geometry changes. By calculating operating speeds at all points along a roadway segment, all operating speeds can be compared, and it can be determined if there is consistency among the operating speeds.

Table 15. Equations used to calculate vehicle operating speeds.

No.	Study	Year	Equation	R^2	Conditions/Notes
1	Fitzpatrick et al.	2000a	$V_{85} = 106.30 - \dfrac{3595.29}{R}$	0.92	Horizontal curve on grade: $0 \leq G < 4$ or horizontal curve combined with sag vertical curve
2	Fitzpatrick et al.	2000a	$V_{85} = 96.46 - \dfrac{2744.49}{R}$	0.56	Horizontal curve on grade: $4 \leq G < 9$
3	Fitzpatrick et al.	2000a	$V_{85} = 100.87 - \dfrac{2720.78}{R}$	0.59	Horizontal curve on grade: $-9 \leq G < 0$
4	Fitzpatrick et al.	2000a	$V_{85} = 101.90 - \dfrac{3283.01}{R}$	0.78	Horizontal curve combined with limited sight distance crest vertical curve
5	Fitzpatrick et al.	2000a	$V_{85} = 111.07 - \dfrac{175.98}{K}$	0.54	Vertical crest curve with limited sight distance on horizontal tangents
6	Fitzpatrick et al.	2000a	$V_{85} = 100.19 - \dfrac{126.07}{K}$	0.68	Sag vertical curve on horizontal tangent

V_{85} = 85th-percentile operating speed (km/h) (1 km/h = 0.621371 mph); R = radius; and K = rate of vertical curvature.

IHSDM can be used to evaluate the design consistency of a roadway. The statistical models developed by Fitzpatrick et al. (2000a, 2000b) are also the basis for the IHSDM DCM. The statistical models developed by Banihashemi et al. (2011) are also incorporated into the IHSDM DCM for lower speed roadways. If using a software program, the user should have the ability to input the roadway design into the software. Typically, the design can be entered manually into the software. Either design plans or field measurements can be referenced to input the design. The IHSDM DCM estimates the 85th-percentile operating speed profiles and applies the design consistency criteria to each direction of travel separately. IHSDM can create a report indicating the level of design consistency along the alignment. Sections with poor consistency (i.e., design inconsistencies) are identified.

Operating speed consistency between adjacent elements may have an association with traffic safety and crash experience (i.e., the greater the differential in operating speed between adjacent horizontal design elements, the higher the expected crash risk). (Wu et al. 2013) If large inconsistencies in anticipated operating speeds are identified, the designs should be modified to produce operating speeds that are more consistent among successive design elements or between the operating speed and designated design speed of the roadway.

Step 3—Set Speed Limit

After one of the design consistency methods is applied, the next step in this design approach is to set a speed limit for the roadway. A speed limit can be set according to statutory speed limits or by using results from the design consistency evaluation (i.e., 85th-percentile operating speed). If a speed limit is to be set according to predicted operating speed, the highest expected operating speed output for the analysis segment should be identified. The posted speed limit should be set

to the highest 5-mph (8.0-km/h) increment that is consistent with the model output. This would imply that the expected operating speed at all speed-influencing features is equal to or lower than the posted speed limit.

METHOD 4—APPLY EXISTING GEOMETRIC DESIGN CRITERIA

Another candidate method to design a self-enforcing or self-explaining two-lane rural highway is to apply existing geometric design criteria so that operating speeds are consistent with designated design speeds. Current geometric design methods, such as those from the AASHTO Green Book, rely on minimum or limiting values; there are no maximum values offered. (AASHTO 2011) Because the Green Book also recommends using larger than the minimum values for different design elements, resulting operating speeds often exceed designated design speeds. This is likely the result of inferred design speeds exceeding the designated design speed, and as a result, the geometric elements have little influence on driver speed choice. Based on existing operating speed models, the use of minimum or near-minimum criteria do appear to be associated with lower driving speeds, while conservatively applying the design criteria produces higher operating speeds.

As such, establishing an upper limit to current geometric design criteria may help produce operating speeds that are consistent with designated design speeds and posted speed limits. When designing roadways, the values recommended in the Green Book can be used as the minimum values for the geometric elements. The upper limit for design criteria that are influenced by the designated design speed can be determined using operating speed model equations. Figure 40 depicts various operating speed model equations by Fitzpatrick et al. (2000a), represented by the solid lines, and the design speed values from the 2011 Green Book, denoted by the dashed lines. While the operating speed models developed by Fitzpatrick et al. (2000a) are used in figure 40, other 85th-percentile operating speed models can be used, such as those shown in chapter 4 or appendix A of this guidance report.
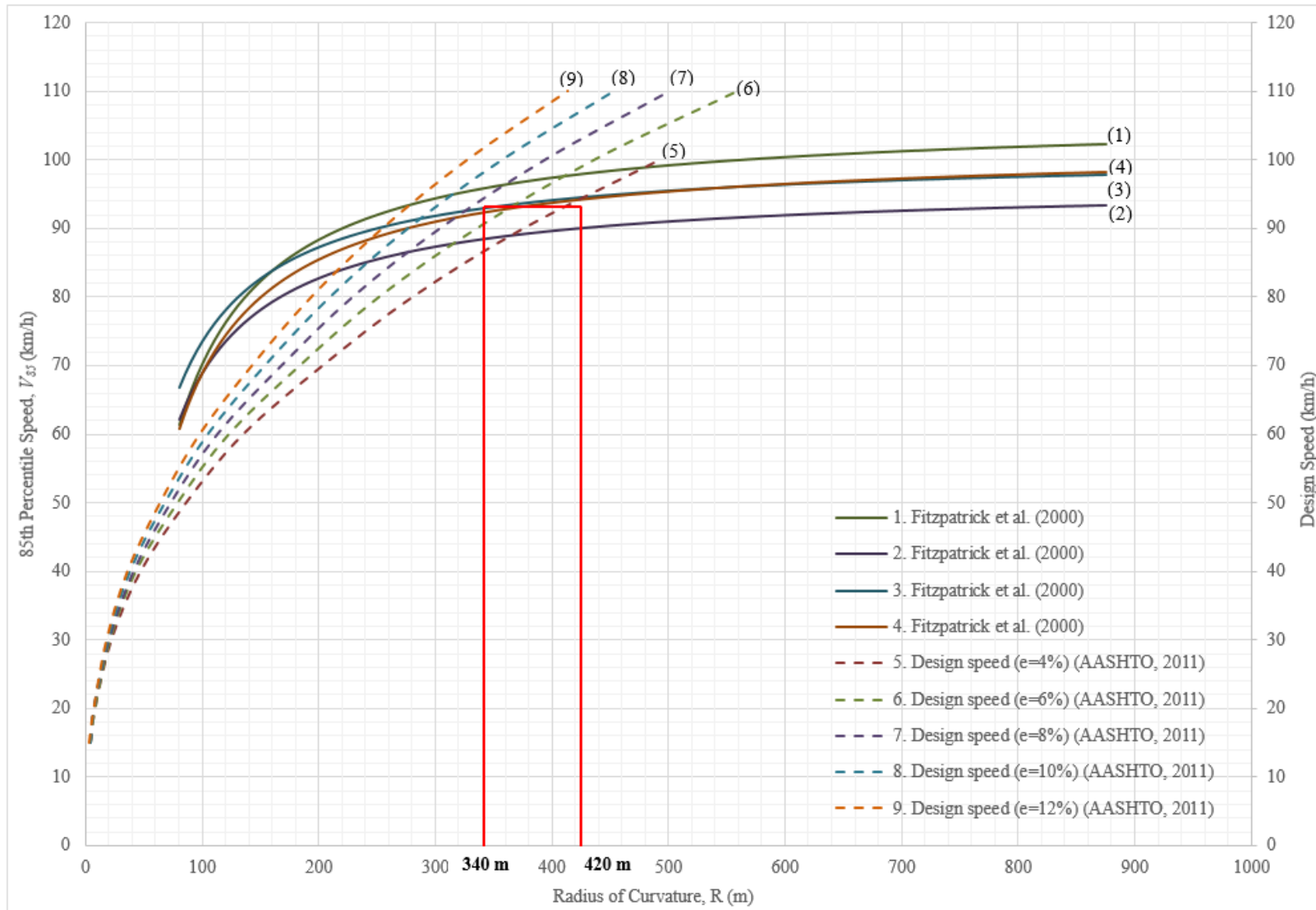
Source: FHWA.
Note: 1 km/h = 0.621371 mph; 1 m = 3.28 ft.

Figure 40. Graph. Ranges of radius of curvature for 85th-percentile and design speeds.

As shown in figure 40, there is a range of values for the radius of curvature that produce the same 85th-percentile operating speeds. An example is highlighted using the red lines in figure 40. The 85th-percentile operating speed equation shown for the example is an equation produced by Fitzpatrick et al. (2000a) to predict speeds on horizontal curves combined with limited sight distance crest vertical curves. The design speed used for the minimum values is the value for roadways with a superelevation of 8 percent from the 2011 Green Book. In this example, the design speed and the target 85th-percentile speed are 58.41 mph (94 km/h) (interpreted using figure 40). Using the Green Book–recommended values for the minimum and the 85th-percentile operating speed equation for the maximum value, an operating speed of 58.41 mph (94 km/h) can be achieved using a radius of curvature ranging from 1,115.49 to 1,377.95 ft (340 to 420 m) for roadways with horizontal curves combined with limited sight distance crest vertical curves. For roads with a superelevation of 8 percent, the Green Book recommends not using a radius of curvature smaller than 1,115.49 ft (340 m). Using a radius of curvature larger than 1,377.95 ft (420 m) may produce 85th-percentile operating speeds larger than the target speed of 58.41 mph (94 km/h).

This process can be used for roadways with different superelevations, vertical grades, and sight distances, or combinations of horizontal and vertical curves. The designated design speeds from the Green Book are different for roads with different maximum rates of superelevation. The operating speed models estimated by Fitzpatrick et al. (2000a), illustrated in figure 40, can be applied for a variety of conditions, such as different vertical grades or various combinations of vertical and horizontal curves. Comparing the designated design speed to the expected operating speed based on a set of geometric features may produce speed harmony for planned roadways.

A candidate framework for applying geometric design criteria as a method to produce self-enforcing roadways is shown in figure 41.
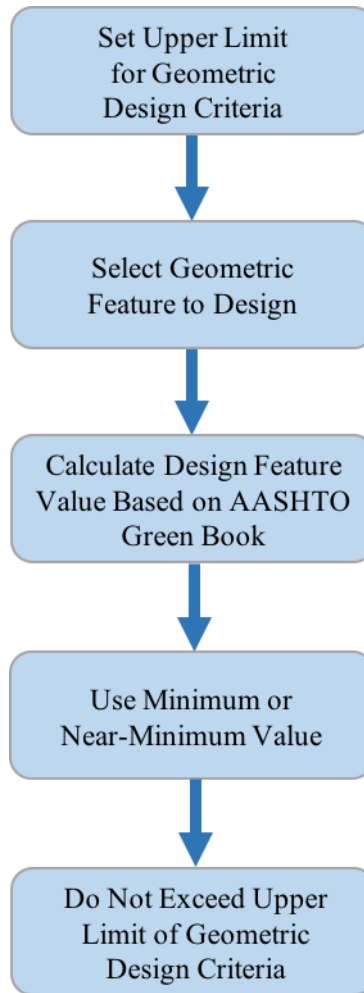
Source: FHWA.

Figure 41. Flowchart. Framework for applying geometric design criteria.

METHOD 5—COMBINATION OF SIGNS AND PAVEMENT MARKINGS

Drivers use visual cues along the roadway to make decisions regarding steering, braking, and throttle inputs. Regarding self-enforcing roadways, drivers using the roadway context to select an operating speed. While there are many factors that affect speed choice, some of these factors include cross-section dimensions, horizontal and vertical alignment, visibility of pavement markings, presence and visibility of signs, posted speed limit, ambient weather conditions, and enforcement presence. (Shinar 2007) Smiley (2016) noted that the primary focus of driver attention is speed control, while a secondary focus is responding to signs.

While signs and pavement markings are important driver decisionmaking inputs, the "human capacity to process information is limited." (Smiley 2016) Consistent road designs allow drivers to "accurately predict the correct path while using minimal visual information processing capacity" in addition to reducing the workload of the driver. (Fitzpatrick et al. 2000b) Information portrayed to drivers needs to be limited, consistent, and displayed in recognizable

patterns, while visual clutter should be minimized and displayed in small chunks to be effective. (Smiley 2016)

A combination of traffic signs and pavement markings can be applied to existing roadway facilities to support the self-enforcing road concepts. A combination of signs and pavement markings can be used to help reduce speeds as a retrofit for existing roadways when reconstructing the road is not feasible. Signs and pavement markings communicate information to drivers regarding speed choice and may encourage drivers to select speeds that are more in harmony with the posted speed limit on the roadway. Many signs and pavement markings have been evaluated and found to affect speeds, and possibly reduce speeds. No published research exists that outlines a process that practitioners may use to implement signs and pavements markings as speed management tools. However, table 16 shows various treatments and combinations of treatments from a study by Boodlal et al. (2015) and indicates how each traffic control device is associated with operating speeds. If there are certain conditions under which the traffic control device should be applied, such as certain geometric conditions, they are noted in table 16.

Table 16. Sign and pavement marking roadway treatments that possible affect speeds. (Boodlal et al. 2015).

Treatment	Description	Effectiveness	Image	Image Number and Caption
Add centerline and edge line pavement markings	Edge line pavement markings and painted center median to reduce lane widths	Decrease 85th-percentile speeds 1 mph (1.6 km/h) on rural main roads, 2 mph (3.2 km/h) on rural two-lane roads during day, 1 mph (1.6 km/h) on rural two-lane roads at night, 0.5 mph (0.8 km/h) on rural two-way roads at stop-controlled intersections	Source: FHWA.	Figure 42. Photo. Centerline and edge line pavement markings on road.
Advisory speed limit sign	Yellow sign displaying advisory speed limit for certain conditions that require a speed lower than the posted speed limit	15-percent reduction in 85th-percentile speeds	Source: FHWA.	Figure 43. Photo. Advisory speed limit sign.

Treatment	Description	Effectiveness	Image	Image Number and Caption
Chevron sign	Chevron alignment signs for guidance on horizontal curves	Study 1: 0.7- to 1.28-mph (1.1- to 2.1-km/h) reduction in 85th-percentile speeds on horizontal curves; Study 2: 1.5- to 3.28-mph (2.4- to 5.28-km/h) reduction in 85th-percentile speeds on horizontal curves; Study 3: 1.28-mph (2.1-km/h) reduction in 85th-percentile speeds on horizontal curves	©Iowa State University.	Figure 44. Photo. Chevron signs around a horizontal curve.
Converging chevron marking pattern	White chevrons installed in a series to give drivers the illusion of driving faster	Study 1: potential to reduce 85th-percentile speeds by 11–24 percent; Study 2: 85th-percentile speed reduction by 3 mph (4.8 km/h) from an FHWA report	©Iowa State University.	Figure 45. Photo. Chevron pavement markings on a road.
Enhanced speed limit legend with colored surfacing	Pavement marking in middle of lane displaying speed limit surrounded by colored box	2-mph (3.2-km/h) reduction in 85th-percentile speed on rural main roadways	©Iowa State University.	Figure 46. Photo. Red speed limit pavement marking on road.
Optical speed bars	Transverse marking either perpendicular to edge lines or entire lane, spaced specifically to give drivers perception of driving faster	Reports of reducing 85th-percentile speeds in differing amounts—0 to 5 mph (0 to 8.0 km/h)	©Virginia Center for Transportation Innovation and Research (Arnold and Lantz 2007).	Figure 47. Photo. Optical speed bars on a road.

Treatment	Description	Effectiveness	Image	Image Number and Caption
Red border speed limit sign	Speed limit sign with red border around it	3-mph (4.8 km/h) reduction in 85th-percentile speed on two-lane highways	©Texas Transportation Institute, 2004.	Figure 48. Photo. Speed limit sign with red border.
SLOW pavement marking legend	SLOW pavement marking legend installed in middle of travel lane	Study 1: 1-mph increase in 85th-percentile speeds; Study 2: 2-mph (3.2-km/h) reduction of 85th-percentile speeds	©Iowa State University.	Figure 49. Photo. SLOW pavement marking in middle of lane on a road before a horizontal curve.
Speed feedback sign	Dynamic signs displaying the operating speed of vehicles	Reduction of 85th-percentile speeds 2 to 7 mph (3.2 to 11.3 km/h)	©Iowa State University.	Figure 50. Photo. Dynamic speed feedback sign on the side of a road.
Speed limit pavement marking legend	Pavement marking in middle of lane displaying speed limit	1-mph reduction in 85th-percentile speeds on rural main roadways	©Iowa State University.	Figure 51. Photo. White speed limit pavement marking in middle of lane.
Speed-activated speed-limit-reminder sign	Sign that is speed activated by approaching vehicles that display speed limit	5-mph (8.0-km/h) reduction in 85th-percentile speed on major roads	©Pennsylvania State University.	Figure 52. Photo. Speed-activated speed-limit-reminder sign.

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Treatment	Description	Effectiveness	Image	Image Number and Caption
Speed-activated warning sign	Warning sign that is speed activated when speed limit is exceeded. The speed activated sign says YOU ARE SPEEDING IF FLASHING	Study 1: 1-mph (1.6 km/h) reduction in 85th-percentile speeds on interstate rural curves; Study 2: 1.6- to 4.7-mph (2.3- to 7.6-km/h) reduction in 85th-percentile speeds on multilane highways; Study 3: 3-mph (4.8-km/h) reduction in 85th-percentile speeds on rural four-lane divided highways	©Transportation Research Board, 2007.	Figure 53. Photo. YOU ARE SPEEDING IF FLASHING sign on side of road.
Transverse markings	Painted raised or flat white transverse markings across center of a travel lane	Study 1: 0.2-mph (0.3-km/h) reduction in 85th-percentile speed on rural horizontal curves; Study 2: 1.4- to 3.9-mph (2.3- to 6.3-km/h) reduction in 85th-percentile speeds on rural highways; Study 3: 3- to 10-mph (4.8- to 16.1-km/h) reduction in 85th-percentile speeds on rural highways; Study 4: 4-mph (6.4-km/h) (11-percent) reduction in 85th-percentile speeds in work zones	©Virginia Center for Transportation Innovation and Research, 2007.	Figure 54. Photo. Transverse pavement markings painted on travel lanes.
Transverse pavement markings with speed feedback sign	Transverse pavement markings in combination with speed feedback signs	4-mph (6.4-km/h) reduction in 85th-percentile speeds	Source: FHWA.	Figure 55. Photo. Transverse pavement markings painted on travel lane.

Treatment	Description	Effectiveness	Image	Image Number and Caption
Variable speed limit sign	Dynamic message sign displaying information related to speed limit	Study 1: 4.7- to 8-mph (7.6- to 12.9-km/h) reduction in 85th-percentile speeds; Study 2: 5-mph (8.0 km/h) reduction in 85th-percentile speeds; Study 3: 0.47- to 0.75-mph (0.8- to 1.2-km/h) reduction in 85th-percentile speeds per every 1-mph (1.6-km/h) reduction in posted speed limit	Source: FHWA.	Figure 56. Photo. Dynamic variable message sign displaying speed limit and traffic information.
Zigzag pavement markings	Zigzag pavement markings painted in travel lane to indicate horizontal curves or crosswalks	1.3-mph (2.1-km/h) reduction in 85th-percentile speeds in suburban areas	©Virginia Center for Transportation Innovation and Research (Dougald 2007).	Figure 57. Photo. Zigzag pavement markings painted on a lane.

METHOD 6—SETTING RATIONAL SPEED LIMITS

Another approach to develop self-enforcing, or self-explaining, roadways is to set speed limits that are reasonable, rational, and consistent with the features of the roadway. A Web-based tool that can be used to provide guidance regarding appropriate posted speed limits for all road types is USLIMITS2, which was produced by FHWA. (FHWA 2016b) The tool determines rational speed limits through expert knowledge of speed limits and a series of decision rules and procedures applied to a particular scenario.

A proposed framework to establish rational speed limits is shown in figure 58.
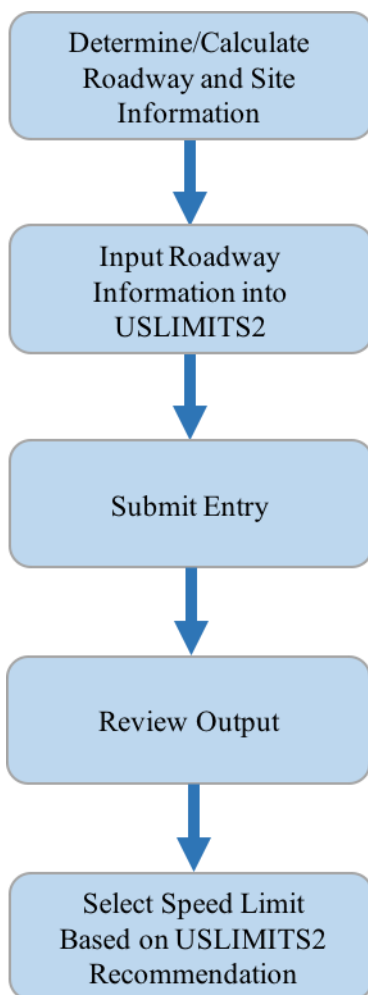
Source: FHWA.

Figure 58. Flowchart. Framework for setting rational speed limits.

The process of using USLIMITS2 is described in the steps below.

Step 1—Determine Roadway Information

The first step in using USLIMITS2 is to identify and calculate all necessary roadway and site information to be inputted into the software. The information provided below should be determined for each roadway where a recommended speed limit is desired. There is general and specific information required to use the program; rational posted speed limits are offered for limited-access freeways, road sections in undeveloped areas, and road sections in developed areas. Road sections in undeveloped areas are the most appropriate category within USLIMITS2 for rural two-lane highways. All necessary information is provided as follows: (FHWA 2014)

- All roadway types:
  - Area and user information: State, county, city/area, user name.
  - Road information: route/street name, begin and end route termini, new or existing route, route type (limited-access freeway, road section in underdeveloped area, road section in developed area).
  - Project information: project date, project/file name, project number, project description.
  - Crash data (if available): years, months, average daily traffic for the period (vehicles/day), total number of crashes for the period, and total number of injury and fatal crashes for the period.

- Limited-access freeway:
  - 85th-percentile speed.
  - 50th-percentile speed.
  - Section length in miles.
  - AADT.
  - Adverse alignment.
  - Statutory speed limit for this type of road.
  - Terrain (flat, rolling, mountainous).
  - Transition zone.
  - Number of interchanges within this section.

- Road section in undeveloped area:
  - 85th-percentile speed.
  - 50th-percentile speed.
  - Section length in miles.
  - AADT.
  - Adverse alignment.
  - Statutory speed limit for this type of road.
  - Transition zone.
  - Roadside rating.
  - Divided/undivided.
  - Number of through lanes.

- Road section in developed area:
  - 85th-percentile speed.
  - 50th-percentile speed.
  - Section length in miles.
  - AADT.
  - Adverse alignment.
  - Statutory speed limit for this type of road.
  - One-way street.
  - Divided/undivided.
  - Number of through lanes.
  - Area type (residential-subdivision, residential-collector, commercial, large complexes).
  - Total number of driveways and unsignalized access points in the section.

- o Total number of signals in the section.
- o On-street parking and usage.
- o Pedestrian/bicycle activity.

Step 2—Input Information into Program and Obtain Output

Once all area and roadway information has been identified and calculated, the next step in the process is to input all relevant roadway information into the USLIMITS2 Web software. As noted earlier, the information is specific to each roadway type, and the software indicates a description of the information required in each field of the program.

After the required information is inputted and the project entry is submitted, USLIMITS2 produces a Speed Zoning Report that provides a summary of the basic project; roadway, crash data, and traffic information; and a recommended speed limit for the roadway. The more information provided in the software, the more accurate the recommended speed limit will be and the more representative it will be of the actual site conditions.

Step 3—Set Speed Limit

After obtaining output from the USLIMITS2 software, the next step in the procedure to set a rational speed limit is to select a posted speed limit. The posted speed limit can be set according to either a statutory speed limit or according to the recommended speed limit from USLIMITS2. USLIMITS2 does take into consideration the statutory speed limit of the area along with other roadway, crash, and traffic information. Setting a speed limit in accordance with the recommendation from USLIMITS2 can produce a more self-enforcing roadway than using only a statutory speed limit.

The six methods presented in this section are candidate processes that may lead to self-enforcing roadways. Some of the methods have similar concepts. For example, the speed feedback loop process, the design consistency method, and the application of geometric design criteria all consider operating speed models. The speed feedback loop process and the design consistency method are both based in design consistency principles.

CHAPTER 5. CASE STUDY EXAMPLES

The following case studies seek to illustrate the relationship between the inferred design speed, designated design speed, and operating speed on two-lane rural highways and describe the use of the various self-enforcing roadways design methods. These methods include the inferred design speed approach, the IHSDM DCM, and USLIMITS2. The other two methods described in the previous section—applying geometric design criteria and employing a combination of signs and pavement markings—will not be applied to the case study examples. The establishment of limiting values for all speed-based geometric design criteria has not been thoroughly evaluated due to limited empirical research on this topic. As such, this method is not applied to the case study examples. The combination of signs and pavement markings method must be applied to roadways, and operating speeds before and after implementation should be collected and analyzed to assess the effectiveness of the markings.

Each of the methods in this section of the report is demonstrated using two case study examples. The first example, US Route 6 in Pennsylvania, illustrates speed harmony on a self-enforcing roadway. The second example, SR 865 in Virginia, demonstrates speed discord and, therefore, is not considered a self-enforcing roadway. The case studies also include information about the roadway characteristics (e.g., horizontal and vertical alignment data, cross-section information, and traffic control devices) and measured operating speeds collected at the sites. The following sections illustrate the self-enforcing road-design approaches for the two case studies.

US ROUTE 6, SHEFFIELD (WARREN COUNTY), PENNSYLVANIA

The US Route 6 study segment is a two-lane, rural principal arterial in Sheffield, Pennsylvania, approximately 1.4 mi (2.3 km) in length. Figure 59 is a map of the study segment, and figure 60 is a plan view of the study segment with the speed data collection locations noted. Data were collected in the eastbound direction of travel using on-pavement sensors. The segment includes seven simple horizontal curves with radii ranging from 716 to 1,432 ft (218.2 to 436.5 m) and five access points, three of which are low-volume, unpaved driveways; the other two access points serve a natural gas plant located on both sides of the highway at the east end of the segment. There are also five crest vertical curves. When the roadway was built in 1925, superelevation was provided on horizontal curves, although no design speed was designated. A subsequent project (completed in 2004) employed a designated design speed of 60 mph (96.6 km/h) and increased the superelevation at each horizontal curve. The vertical alignment has not been altered since construction in 1925. The available sight distance for one crest vertical curve located near the middle of the study segment is less than the criteria associated with a 60-mph (96.6 km/h) design speed. The maximum grade within the segment is 7 percent. The posted speed limit is 55 mph (88.5 km/h), but there are no speed limit signs within the study segment.

As shown in figure 61, the typical cross section includes one travel lane in each direction and a paved shoulder. The clear zone is narrow with a guardrail adjacent to the shoulder along much of the eastbound side. The adjacent land is wooded with a natural gas plant located at the eastern end of the study segment. There are no designated pedestrian facilities in the study segment. Signs indicate that the roadway is a bicycle corridor. The AADT for the study segment is

approximately 2,500 vehicles/d, of which 10 percent is heavy vehicles. There were no observed pedestrians or bicycle traffic during the data collection period.
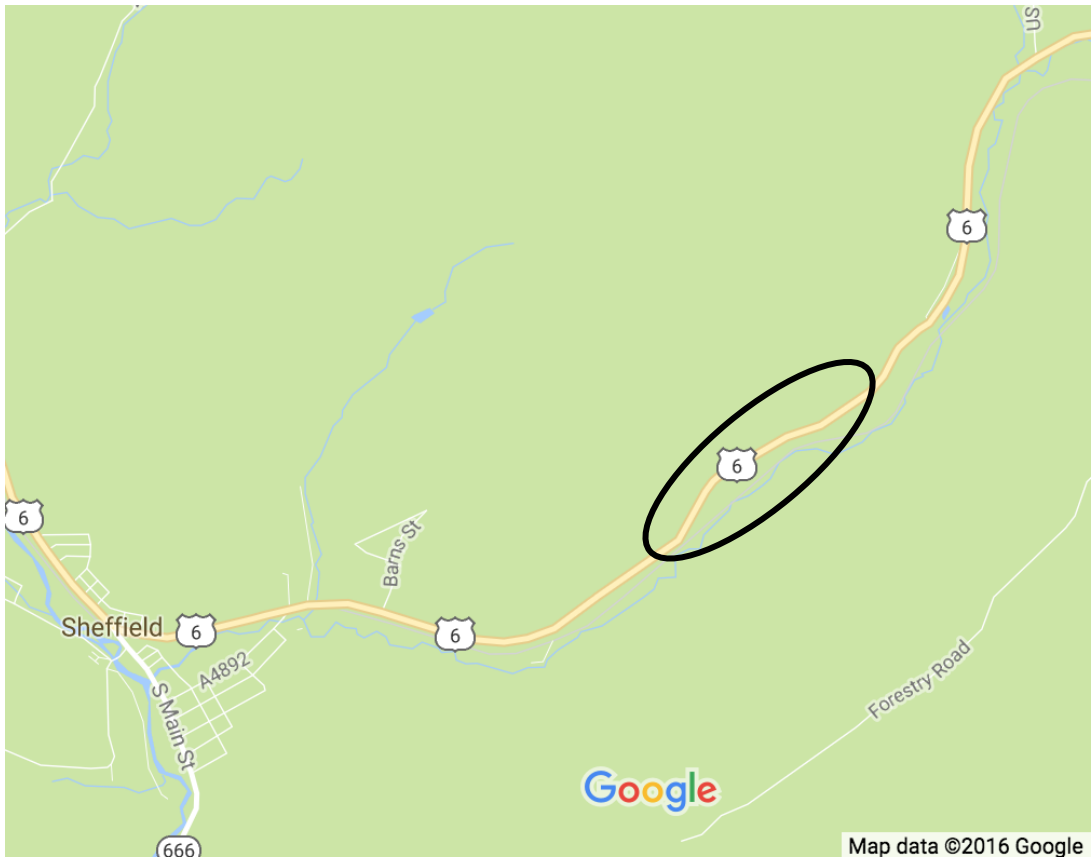
©2016 Google®; annotations by The Pennsylvania State University and Institute of Transportation Engineers.

Figure 59. Map. US Route 6 study segment map.

CURVE 2
PI STA.17+25.055
Δ-34°48'00" RT.
D-4°00'
T-448.89'
L-870.00'
R-1432.39'
E-68.69'
S.E.-4.8%

CURVE 4
PI STA.42+82.880
Δ-19°04'00" LT.
D-6°00'
T-160.37'
L-317.78'
R-954.93'
E-13.37'
S.E.-6.8%

CURVE 6
PI STA.73+04.88
Δ-34°18'00" RT.
D-4°30'
T-392.92'
L-762.22'
R-1273.24'
E-59.25'
S.E.-5.1%

PC STA.0+0.000
PT STA.5+10.000
PC STA.12+90.070
PT STA.21+60.040
PC STA.35+34.070
PT STA.38+85.320
PC STA.54+51.520
PT STA.61+89.850
PC STA.74+83.220
PT STA.78+35.720

CURVE 1
PI STA.2+55.000
Δ-30°36'00" LT.
D-6°00'
T-261.24'
L-510.00'
R-954.93'
E-35.09'
S.E.-6.1%

CURVE 3
PI STA.37+09.695
Δ-14°03'00" RT.
D-4°00'
T-176.51'
L-351.25'
R-1432.39'
E-10.83'
S.E.-5.8%

CURVE 5
PI STA.58+20.685
Δ-29°32'00" LT.
D-4°00'
T-377.56'
L-738.33'
R-1432.39'
E-48.93'
S.E.-4.8%

CURVE 7
PI STA.76+59.470
Δ-28°12'00" LT.
D-8°00'
T-179.90'
L-352.50'
R-716.20'
E-22.25'
S.E.-6.6%

PC STA.41+23.990
PT STA.44+41.770
PC STA.65+42.660
PT STA.73+04.880

Source. FHWA.

Figure 60. Illustration. US Route 6 plan view.
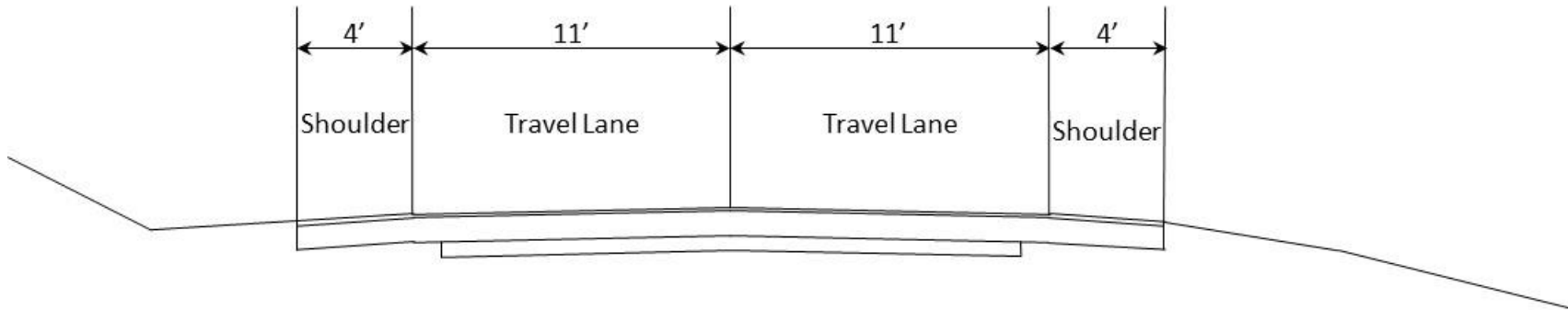
Source: FHWA.

Figure 61. Illustration. US Route 6 typical cross section.

Inferred Design Speed Approach

The inferred design speed approach was used to create an inferred design speed plot and compare the designated design speed, posted speed limit, and 85th-percentile operating speeds based on the field data collected. Figure 62 is the generated speed profile for the study segment (direction of travel is left to right). At numerous locations, sight distances are limited by the vertical and horizontal alignment combined with lateral obstructions, mostly cut slopes. Sight distance restrictions translate to inferred design speeds as low as 39 mph (62.8 km/h). The inferred design speeds for horizontal curves range from 48 to 61 mph (77.2 to 98.2 km/h). A maximum inferred design speed was used at some locations, typically along tangents. On these sections, there are no geometric features to limit operating speeds, which could be high. The vertical axis of the speed profile plot shown in figure 62 was truncated at 80 mph (128.7 km/h) to show the variability in the operating speeds along the entire study segment and to fit the speed profile on a single page. An advisory speed of 40 mph (64.4 km/h) is posted at four horizontal curves and an advisory speed of 50 mph (80.5 km/h) is posted at two other curves. The advisory speeds are shown on the speed profile plot along with the 85th-percentile operating speeds for each successive geometric element (horizontal curve and tangent).
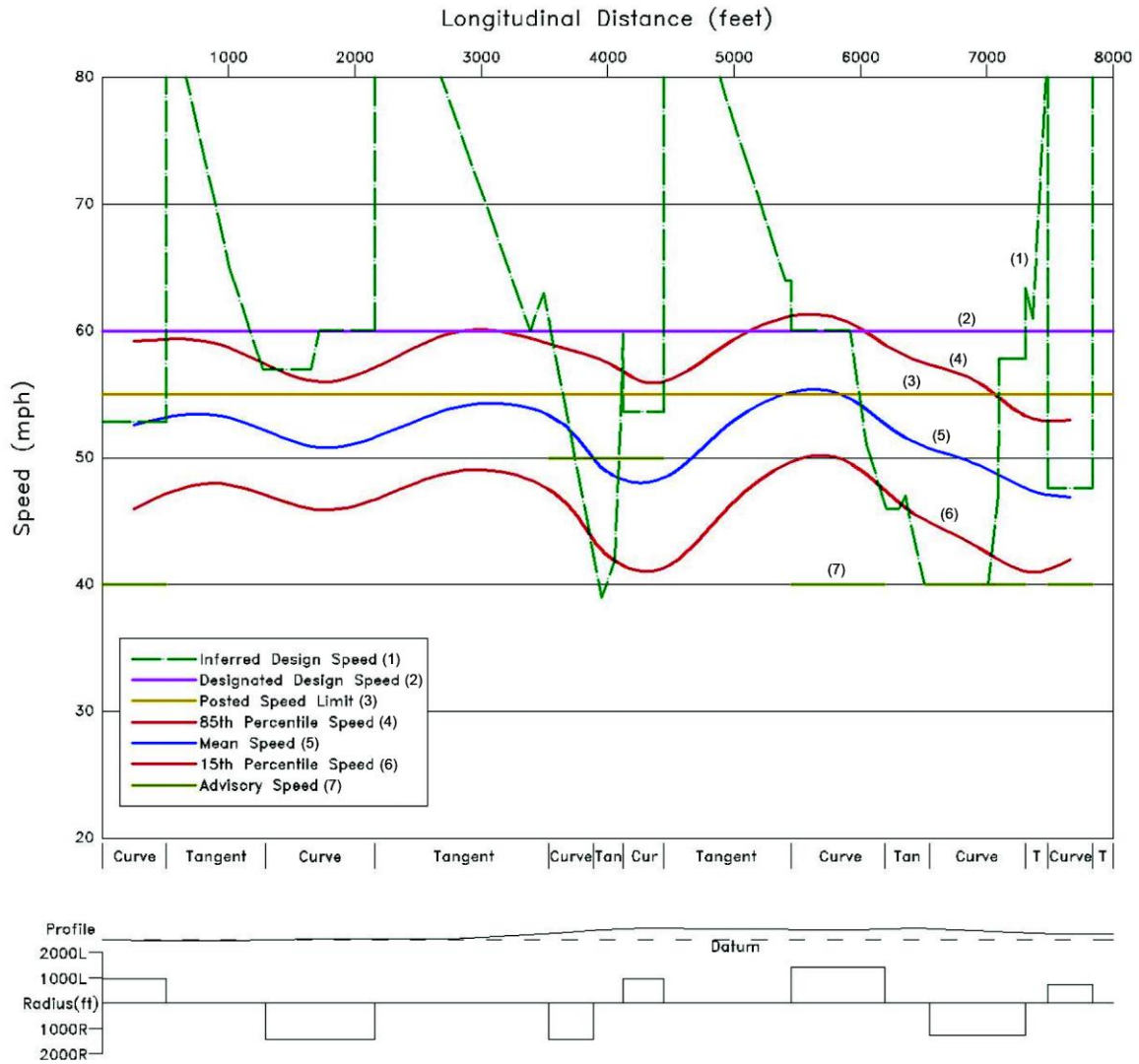
Source: FHWA.
Note: 1 mph = 1.60934 km/h.

Figure 62. Graph. US Route 6 speed profile.

As shown in figure 62, speed harmony generally exists between the posted speed limit, designated design speed, and 85th-percentile operating speeds. Speed harmony is an example of a self-enforcing roadway. The 85th-percentile operating speed exceeds the designated design speed at only one location, a horizontal curve. The measured speeds on this horizontal curve are higher than on the approach tangent. The inferred design speed is well below the designated design speed at two locations due to SSD restrictions. Sight distance is limited at station 40+00 by a crest vertical curve and by lateral obstructions between stations 60+00 and 70+00, which results in the 85th-percentile speeds exceeding the inferred design speed by 15 to 20 mph (24.1 to 32.2 km/h).

IHSDM DCM

The IHSDM DCM was used to identify any possible speed inconsistencies for the US Route 6 case study. The operating speed model can help determine any potential speed management or safety issues along the roadway. Data input to the IHSDM include speed information (e.g., desired speed, design speed, and posted speed limit for lower speed highways), horizontal curvature information, vertical curvature information, and information on the surrounding area type (e.g., rural, suburban, or urban). Figure 63 shows the IHSDM DCM output. The US Route 6 geometric elements, desired speed, design speed, and 85th-percentile operating speeds are included in the plot.

NOTE: Speed profile does NOT account for intersections.

Legend
Intersections (1)
Vertical Alignment (Elevation); ft (2)
Vertical Alignment Curvature K Value; ft/% (3)
Horizontal Alignment Degree of Curve; deg (4)
Horizontal Alignment Radius; ft (5)
Posted Speed; mph (6)

Desired Speed; mph (7)
Design Speed; mph (8)
V85 Speed; mph (9)
design element differential < 6 MPH (10)
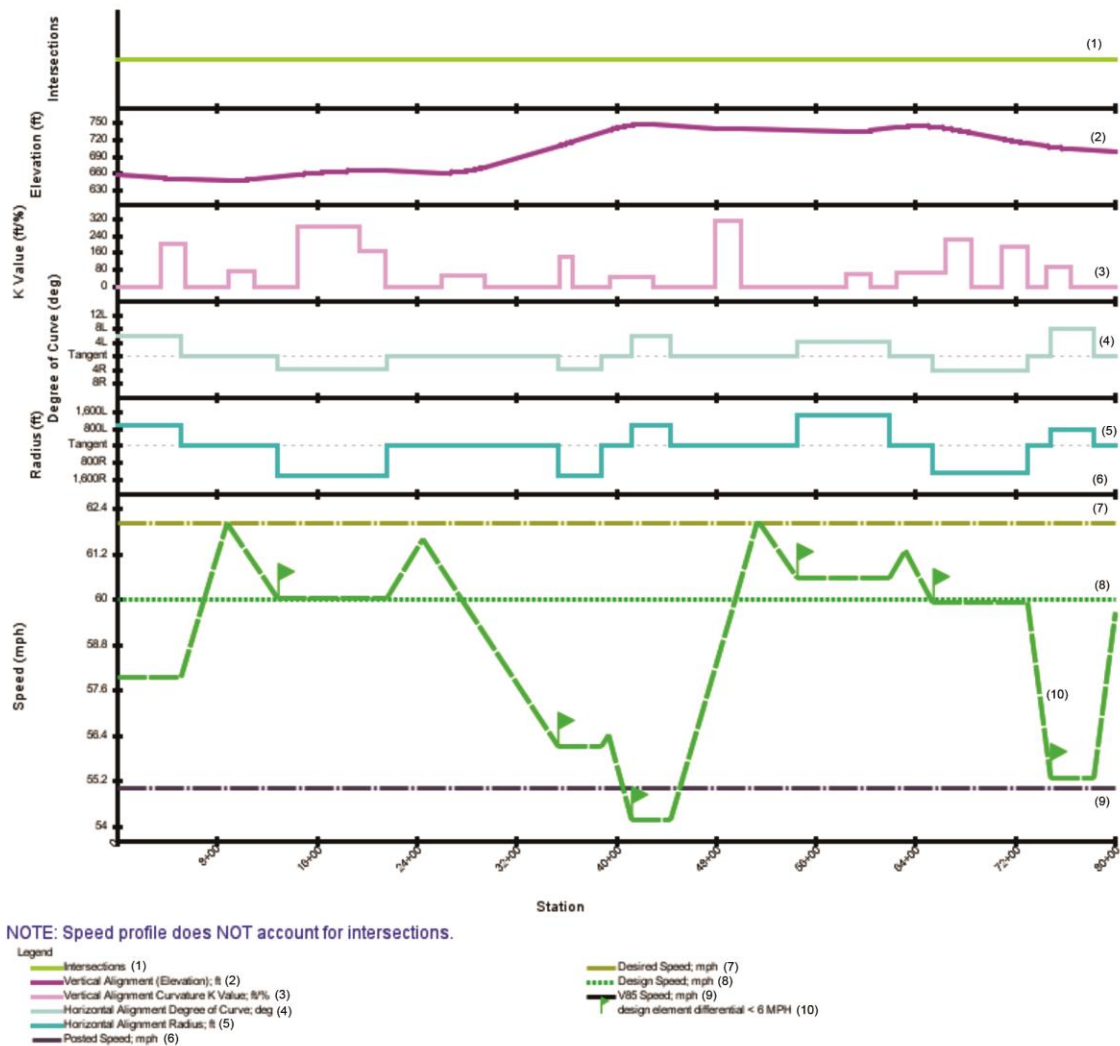
Source: FHWA.
Note: 1 mph = 1.60934 km/h.

Figure 63. Illustration. US Route 6 IHSDM output.

In figure 63, the green flags show that the speed differential between adjacent design elements is less than 6 mph (9.7 km/h). This indicates consistency among adjacent design elements; no sharp

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speed reductions between elements are required. Table 17, table 18, and table 19 display the results from the IHSDM DCM in tabular format.

Table 17. US Route 6 58th-percentile speed profile coordinates.

Station	Segment Type	V_{85} Speed (mph (km/h))	Speed Model
0.000	Curve	58 (93.3)	High-speed
5+10.000	Non-curve	58 (93.3)	High-speed
8+81.829	Non-curve	62 (99.8)	High-speed
12+90.070	Curve	60 (96.6)	High-speed
21+60.040	Non-curve	60 (96.6)	High-speed
24+53.628	Non-curve	62 (99.8)	High-speed
35+34.070	Curve	56 (90.1)	High-speed
38+85.320	Non-curve	56 (90.1)	High-speed
39+38.298	Non-curve	56 (90.1)	High-speed
41+23.990	Curve	54 (86.7)	High-speed
44+41.770	Non-curve	54 (86.7)	High-speed
51+36.439	Non-curve	62 (99.8)	High-speed
51+58.050	Non-curve	62 (99.8)	High-speed
54+51.520	Curve	61 (98.2)	High-speed
61+89.850	Non-curve	61 (98.2)	High-speed
63+23.880	Non-curve	61 (98.2)	High-speed
65+42.660	Curve	60 (96.6)	High-speed
73+04.880	Non-curve	60 (96.6)	High-speed
74+83.220	Curve	55 (88.5)	High-speed
78+35.720	Non-curve	55 (88.5)	High-speed
80+00.000	Non-curve	60 (96.6)	High-speed

Table 18. US Route 6 design speed assumption.

From Station	To Station	Min (mph (km/h))	Max (mph (km/h))	Condition
0.000	6+98.956	–2 (–3.2)	0 (0)	4
6+98.956	27+67.203	0 (0)	2 (3.2)	1
27+67.203	49+59.326	–6 (–9.7)	0 (0)	4
49+59.326	65+28.512	0 (0)	2 (3.2)	1
65+28.512	80+00.000	–5 (–8.0)	0 (0)	4

Condition 1 = 0 mph (0 km/h) ≤ ($V_{85} - V_{design}$) ≤ 6 mph (9.7 km/h);
Condition 4 = ($V_{85} - V_{design}$) < 0 mph (0 km/h); V_{85} = estimated 85th-percentile operating speed;
V_{design} = design speed.

Table 19. US Route 6 speed differential of adjacent design element.

Station of Max Speed on Preceding Element	Max Speed on Preceding Element (mph (km/h))	Start of Curve	Speed on Curve (mph (km/h))	Speed Differential (mph (km/h))	Condition
8+81.829	62 (99.8)	12+90.070	60 (96.6)	2 (3.2)	1
24+53.628	62 (99.8)	35+34.070	56 (90.1)	5 (8.0)	1
39+38.298	56 (90.1)	41+23.990	54 (86.7)	2 (3.2)	1
51+36.439	62 (99.8)	54+51.520	61 (98.2)	1 (1.6)	1
63+23.880	61 (98.2)	65+42.660	60 (96.6)	1 (1.6)	1
65+42.660	60 (96.6)	74+83.220	55 (88.5)	5 (8.0)	1

Condition 1 = 0 mph ($V_{85,Tangent} - V_{85,Curve}$) ≤ 6 mph (9.7 km/h); $V_{85,Tangent}$ = estimated 85th-percentile operating speed on tangent; $V_{85,Curve}$ = estimated 85th-percentile operating speed at the beginning of the curve.

As shown, the speed differential between the estimated 85th-percentile operating speed and the design speed (table 18) and the speed differential between the estimated 85th-percentile operating speed on tangent and the estimated 85th-percentile operating speed at the beginning of the curve (table 19) are all less than 6 mph (9.7 km/h). Similar to figure 63, table 17, table 18, and table 19 also indicate that the posted speed limit, 85th-percentile operating speeds, and design speed are in harmony. Therefore, US Route 6 is considered a self-enforcing, or self-explaining, roadway.

USLIMITS2

The USLIMITS2 Web-based software was used to determine a recommended speed limit for US Route 6 based on various features of the roadway and the surrounding area. Table 20, table 21, and table 22 present the information inputted into USLIMITS2 for US Route 6.

Table 20. US Route 6 USLIMITS2 basic project factor inputs.

Basic Project Factor	Basic Project Information
State	Pennsylvania
County	Warren County
City	Sheffield census-designated place
Route type	Road section in undeveloped area
Route status	Existing

Table 21. US Route 6 USLIMITS2 roadway factor inputs.

Roadway Factor	Roadway Information
Section length	1.4 mi (2.3 km)
Statutory speed limit	55 mph (88.5 km/hr)
Adverse alignment	No
Divided/undivided	Undivided
Number of lanes	2
Roadside hazard rating	3–5*
Transition zone	No

*A roadside hazard rating of 3 and 5 both produce the same recommended speed limit.

Table 22. US Route 6 USLIMITS2 traffic factor inputs.

Traffic Factor	Traffic Information
85th-percentile speed	57 mph (91.7 km/h)
50th-percentile speed	52 mph (83.7 km/h)
AADT	2,500 vehicles/d

Using the inputs in table 20, table 21, and table 22, USLIMITS2 produced a recommended speed limit of 55 mph (88.5 km/h) for the US Route 6 study segment. This is consistent with the actual posted speed limit of US Route 6, which is also 55 mph (88.5 km/h).

Design Consistency of US Route 6

The design of US Route 6 produces operating speeds that are consistent with the posted speed limit and the designated design speed. The geometric design, which contributes to the presence of speed harmony for this roadway segment, is a variation of horizontal and vertical curves, including sight distance restrictions. These geometric features reduce the driver's ability to operate motor vehicles at speeds that exceed the posted or designated design speed. Another respect in which US 6 operating speeds are consistent with the posted speed limit and designated design speed concerns the correlation between the designated design speed and the posted speed limit. The designated design speed and the posted speed limit differ by 5 mph (8.0 km/h). The designated design speed is 60 mph (96.6 km/h), and the posted speed limit is 55 mph (88.5 km/h). Operating speeds tend to fall within this 5-mph (8.0 km/h) gap. The speed limit appears to be set in accordance with the 85th-percentile operating speeds and is a typical limit for the area type surrounding the roads and for a two-lane rural highway.

Safety Performance of US Route 6

Ten yr (2005 through 2014) of crash data for US Route 6 were compiled to assess the historical safety performance of this roadway. The study reported on crashes from the period, with three of them being fatal. Table 23 shows the breakdown per year for the reported crashes occurring on US Route 6.

Table 23. Historical crash data for the US Route 6 study segment.

Year	Total Crashes	Fatal and Injury Crashes
2005	1	1
2006	0	0
2007	0	0
2008	0	0
2009	0	0
2010	1	0
2011	1	1
2012	1	0
2013	4	1
2014	0	0
Total	**8**	**3**

The IHSDM contains a module that predicts crashes based on the geometric elements of the roadway, the surrounding area, and traffic conditions, such as AADT. This crash prediction module was used to predict crashes for the years 2005 through 2014. Crashes were predicted sing the module with and without the empirical Bayes (EB) method. The crash predictions using the EB method were completed using the HSM calibration factor and then using a county-specific calibration factor for District 1 in Pennsylvania, where this site is located. The calibration factor for District 1 in Pennsylvania is 1.05. These predicted numbers were compared to the reported crash data for the same 10-yr period. Table 24 compares the historical reported crash data with the predicted past crashes.

Table 24. Comparison of historical crash data and number of crashes predicted using the IHSDM.

Years	Total Crashes	Fatal and Injury Crashes
Historical 2005–2014	8	3
IHSDM 2005–2014	16.77	5.38
IHSDM with EB method	9.86	3.52
IHSDM with EB method and Pennsylvania District 1 calibration factor	10.353	3.70

As shown in table 24, the predicted number of crashes from the IHSDM is approximately double the number of reported crashes identified through historical crash data. The actual historical crash data indicate that a total of eight crashes (with three being fatal-plus-injury crashes) occurred in 10 yr. This translates to a crash rate (for total crashes) of 0.571 crashes per mile (1.6 km), per year. When the EB method was applied using the calibration factor for District 1 in Pennsylvania, the predicted number of crashes is closer to the reported number of crashes. This analysis and comparison between historical crashes and predicted crashes from the IHSDM shows that speed management is important. In this particular case study, a roadway that exhibits speed harmony has fewer reportable crashes than predicted by the IHSDM crash prediction

module. Because this is only a single case study, additional studies should be undertaken to determine if this relationship exists on rural two-lane highways.

SR 865 (ROCKFISH ROAD), MADRID (AUGUSTA COUNTY), VIRGINIA

The study segment is a nearly 1-mi (1.6-km) section of a two-lane rural major collector. Figure 64 is a map of the study segment location, and figure 65 is a plan view of the study segment with the speed sensor locations noted. Data were collected in the northbound direction of travel. The study segment has five horizontal curves with radii ranging from 477 to 3,418 ft (145.4 to 1,041.8 m). The second and third horizontal curves constitute a reverse curve, and the third and fourth curves are compound curves. The result is a series of three contiguous curves, unseparated by tangents. The designated design speed is 40 mph (64.4 km/h). The posted speed limit is 45 mph (72.4 km/h) in the study segment, with one advisory speed of 35 mph (56.3 km/h) in place for a school zone. The first and last curves on the study segment narrowly exceed the minimum criteria for the designated design speed of 40 mph (64.4 km/h). The three consecutive curves have larger radii, and the inferred design speeds well exceed the designated design speed. State Routes 828 and 865 form a four-way intersection, one leg being the entrance to an elementary school. Left-turn lanes are provided on both Rockfish Road approaches. The average segment access density is 30.6 access points per mile consisting primarily of residential driveways.

The typical cross section of the segment, shown in figure 66, consists of one travel lane in each direction flanked by unpaved earthen shoulders. There are no facilities for pedestrian or bicycle traffic. Detached, single-family homes are moderately spaced along both sides of the alignment and set back over 50 ft (15.2 m) from the roadway. An elementary school is located in the approximate middle of the study segment. The clear zone is about 30 ft (9.1 m) for most of the alignment. The guardrail is located along the first and third horizontal curves and separated from the edge of the traveled way by approximately 10 ft (3.0). The average daily traffic on the study segment is approximately 600 vehicles/d, of which 4 percent is heavy vehicles. There was no observed pedestrian or bicycle traffic during the data collection period. Additionally, operating speeds were collected at consecutive horizontal curves and tangents throughout the study segment.
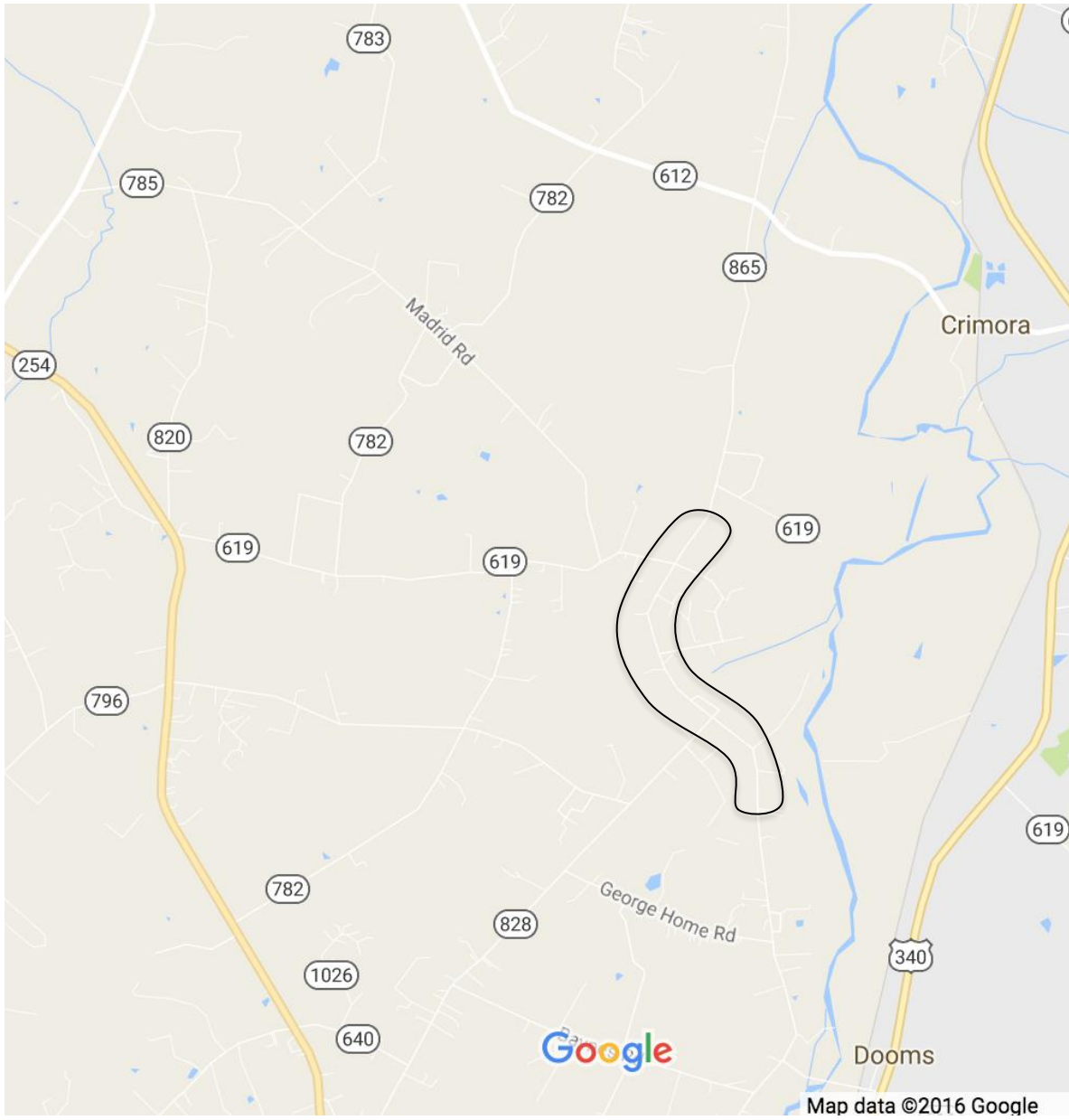
©2016 Google®; annotations by The Pennsylvania State University and Institute of Transportation Engineers.

Figure 64. Map. SR 865 (Rockfish Road) study segment map.

CURVE 2
PI STA.20+01.78
Δ-4°26'54" LT.
D-1°40'
T-132.74'
L-265.35'
R-3417.89'
E-2.29%

CURVE 4
PI STA.28+50.140
Δ-32°55'47" RT.
D-3°30'
T-483.82'
L-940.85'
R-1637.02'
E-8.00% MAX

CURVE 1
PI STA.4+84.335
Δ-49°44'49" LT.
D-12°00'
T-221.36'
L-414.5'
R-477.46'
E-7.99%

CURVE 3
PI STA.22+59.540
Δ-4°26'54" RT.
D-1°46'
T-125.14'
L-250.16'
R-3222.1'
E-2.40%

CURVE 5
PI STA.45+38.740
Δ-52°14'00" RT.
D-12°00'
T-234.08'
L-435.28'
R-477.47'
E-8.00% MAX

PC STA.2+77.040
PT STA.6+91.630
PC STA.18+69.100
PRC STA.21+34.460
PCC STA.23+84.620
PT STA.33+15.660
PC STA.43+21.100
PT STA.47+56.380

8

Figure 65. Illustration. SR 865 (Rockfish Road) plan view.

4'   6'   11'   11'   6'
      Shoulder   Travel Lane   Travel Lane   Shoulder
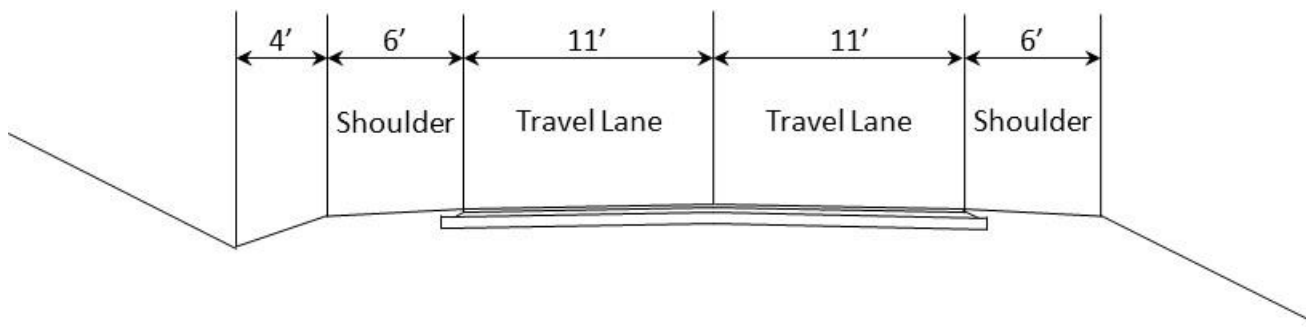
Figure 66. Illustration. SR 865 (Rockfish Road) typical cross section.

Inferred Design Speed Approach

The inferred design speed approach was used to create an inferred design seed plot and compare the designated design speed, posted speed limit, and operating speeds based on field data. Figure 67 shows a speed profile plot for Rockfish Road (direction of travel is left to right).
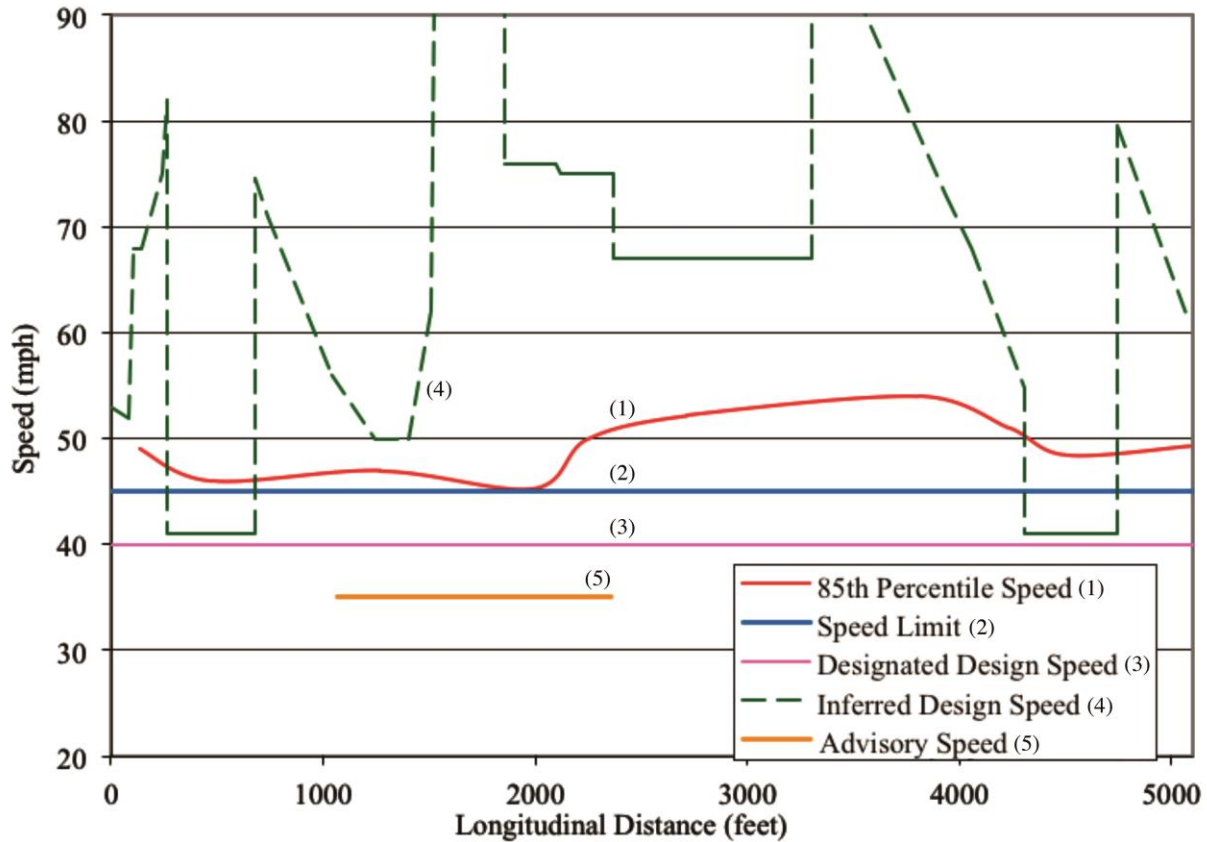
Source: FHWA.

Figure 67. Graph. SR 865 (Rockfish Road) speed profile.

As shown in figure 67, speed discord generally exists between the posted speed limit, designated design speed, and 85th-percentile operating speeds. The geometric design of the SR 865 study segment produces operating speeds that are higher than the posted speed limit. This is not an example of a self-enforcing, or self-explaining, roadway. Inferred design speeds range from 41 to 92 mph (66.0 to 148.1 km/h). The posted speed limit exceeds the designated design speed, and the posted speed limit also exceeds the inferred design speed along two horizontal curves. These two horizontal curves appear to be associated with lower operating speeds when approaching the horizontal curves. The 85th-percentile operating speeds are close to the posted speed limit near the beginning and end of the study segment where the inferred design speeds are near the designated and posted speeds. Observed speeds increase beyond the school zone, which has a posted advisory speed limit of 35 mph (56.3 km/h).

IHSDM DCM

The IHSDM DCM was used to identify any possible speed inconsistencies for SR 865. The operating speed model can help determine any potential design consistency issues along the roadway. The inputs into the IHSDM include speed information (e.g., desired speed and design speed, and the posted speed limit for lower speed highways), horizontal curvature information, vertical curvature information, and information on the surrounding area type (e.g., rural, suburban, or urban). Figure 68 shows the IHSDM DCM output. The SR 865 geometric elements, desired speed, designated design speed, and 85th-percentile operating speeds are shown on the plot.
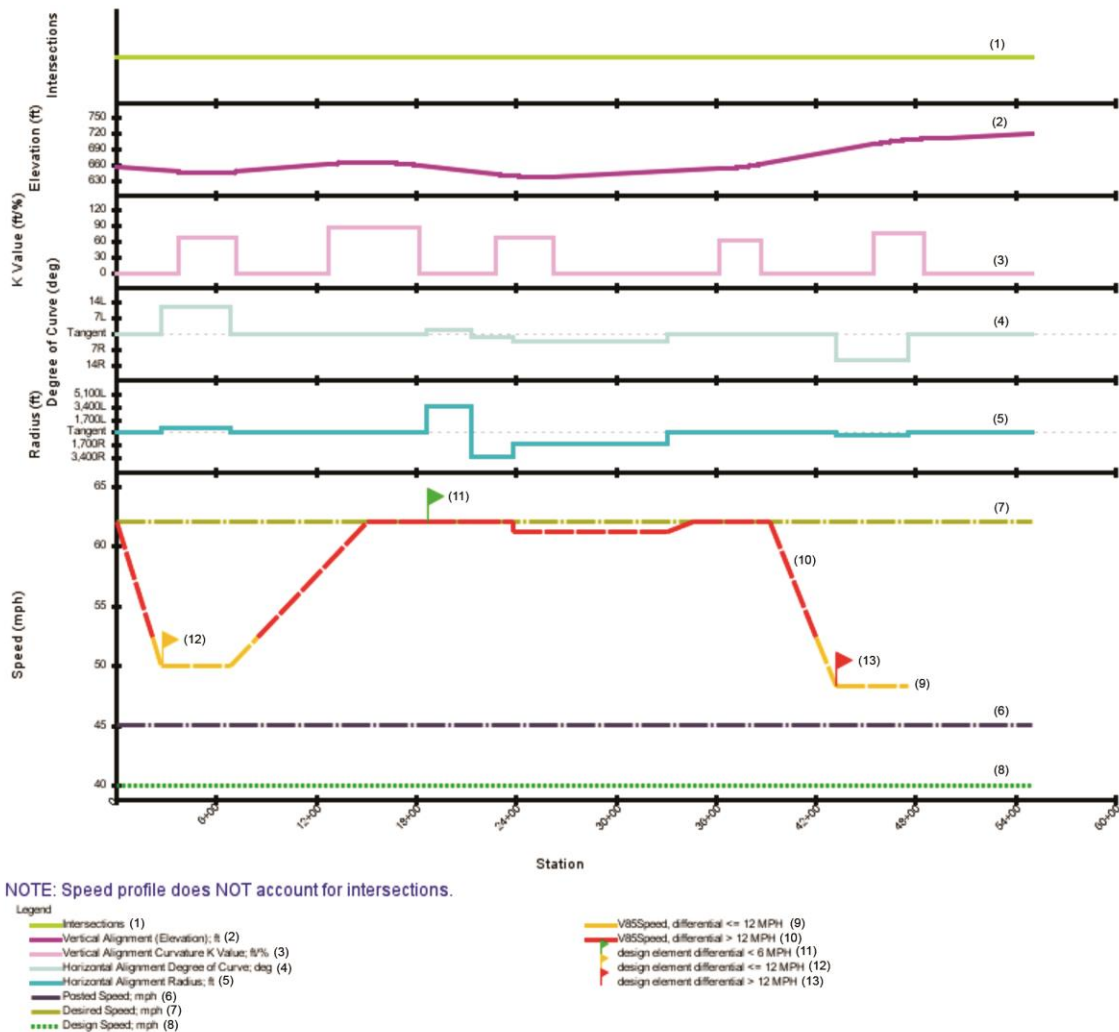
Source: FHWA.
Note: 1 mph = 1.60934 km/h.

Figure 68. Illustration. SR 865 (Rockfish Road) IHSDM output.

In figure 68, the green flag shows that the operating speed differential between adjacent design elements is less than 6 mph (9.7 km/h). The yellow flag indicates that the speed differential between adjacent design elements is greater than or equal to 6 mph (9.7 km/h and less than or

equal to 12 mph (19.3 km/h). The red flag shows that the speed differential between adjacent design elements is greater than 12 mph (19.3 km/h). The presence of yellow and red flags indicates there is no consistency among the geometric elements on the SR 865 case study site.

While the flags in figure 68 represent the design element differential, the color-coded speed profile represents the 85th-percentile operating speed differential, which is the speed differential between the design speed and the 85th-percentile operating speed. The orange portions of the profile line represent speed differentials that are less than or equal to 12 mph (19.3 km/h), and the red portions of the speed profile lines represent speed differentials that are greater than 12 mph (19.3 km/h). Both the red and orange portions of the line indicate that the difference between the design speed and the 85th-percentile operating speed is large, which suggests there is no consistency among the design elements. Table 25, table 26, and table 27 display the results from the IHSDM DCM in tabular format.

Table 25. SR 865 (Rockfish Road) 85th-percentile speed profile coordinates.

Station	Segment Type	V_{85} Speed (mph (km/h))	Speed Model
0.000	Non-curve	62 (99.8)	High-speed
2+77.040*	Curve*	50 (80.5)*	High-speed*
6+91.630	Non-curve	50 (80.5)	High-speed
15+06.790	Non-curve	62 (99.8)	High-speed
18+69.100	Curve	62 (99.8)	High-speed
21+34.458	Curve	62 (99.8)	High-speed
23+84.618	Curve	62 (99.8)	High-speed
23+84.620	Curve	61 (98.2)	High-speed
33+15.660	Non-curve	61 (98.2)	High-speed
34+63.233	Non-curve	62 (99.8)	High-speed
39+24.277	Non-curve	62 (99.8)	High-speed
43+21.100	Curve	48 (77.2)	High-speed
47+56.380	Non-curve	48 (77.2)	High-speed

*The deceleration rate predicted for the range(s): [0.000 to 2+77.040] (in the direction of increasing stations) is greater than the approximated comfortable deceleration rate as determined by data collected to develop the DCM (as referenced in FHWA Report No. FHWA-RD-99-171, *Speed Prediction for Two-Lane Rural Highways*).

Table 26. SR 865 (Rockfish Road) design speed assumption.

From Station	To Station	Min (mph (km/h))	Max (mph (km/h))	Condition
0.000	2+21.233	12 (19.3)	22 (35.4)	3
2+21.233	8+55.836	10 (16.1)	12 (19.3)	2
8+55.836	42+01.139	12 (19.3)	22 (35.4)	3
42+01.139	47+56.380	8 (12.9)	12 (19.3)	2

Condition 2 = 6 mph (9.7 km/h) $< (V_{85} - V_{design}) \leq$ 12 mph (19.3 km/h); Condition 3 = 12 mph (19.3 km/h) $< (V_{85} - V_{design})$; V_{85}-estimated 85th-percentile operating speed; V_{design} = design speed.

Table 27. SR 865 (Rockfish Road) speed differential of adjacent design element.

Station of Max Speed on Preceding Element	Max Speed on Preceding Element (mph (km/h))	Start of Curve	Speed on Curve (mph (km/h))	Speed Differential (mph (km/h))	Condition
0.000	62 (99.8)	2+77.040	50 (80.5)	12 (19.3)	2
15+06.790	62 (99.8)	18+69.100	62 (99.8)	0 (0)	1
34+63.233	62 (99.8)	43+21.100	48 (77.2)	14 (22.5)	3

Condition 1 = 0 mph (0 km/h) ($V_{85,Tangent} - V_{85,Curve}$) ≤ 6 mph (9.7 km/h); Condition 2 = 6 mph (9.7 km/h) < ($V_{85,Tangent} - V_{85,Curve}$) ≤ 12 mph (19.3); Condition 3 = 12 mph (19.3 km/h) < ($V_{85,Tangent} - V_{85,Curve}$); $V_{85,Tangent}$ = estimated 85th-percentile operating speed on tangent; $V_{85,Curve}$ = estimated 85th-percentile operating speed at the beginning of the curve.

As shown, the speed differential between the estimated 85th-percentile operating speed and the designated design speed (table 26), and the speed differential between the estimated 85th-percentile operating speed on tangent and the estimated 85th-percentile operating speed at the beginning of the curve (table 27) are all greater than 6 mph (9.7 km/h). Similar to figure 68, table 25, table 26, and table 27 also indicate there are inconsistencies between the posted speed limit, 85th-percentile operating speeds, and the designated design speed. Speed discord occurs; therefore, SR 865 is not considered a self-enforcing, or self-explaining, roadway.

USLIMITS2

The USLIMITS2 Web-based software was utilized to determine a recommended posted speed limit for SR 865 based on the existing roadway features and the surrounding area.

Table 28, table 29, and table 30 show the information that was inputted into USLIMITS2 for the SR 865 study site.

Table 28. SR 865 (Rockfish Road) USLIMITS2 basic project factor inputs.

Basic Project Factor	Basic Project Information
State	Virginia
County	Augusta County
City	Rural/other
Route type	Road section in undeveloped area
Route status	Existing

Table 29. SR 865 (Rockfish Road) USLIMITS2 roadway factor inputs.

Roadway Factor	Roadway Information
Section length	1 mi (1.6 km)
Statutory speed limit	55 mph (88.5 km/h)
Adverse alignment	No
Divided/undivided	Undivided
Number of lanes	2
Roadside hazard rating	2–3*
Transition zone	No

*A roadside hazard rating of 2 and 3 both produce the same recommended speed limit.

Table 30. SR 865 (Rockfish Road) USLIMITS2 roadway factor inputs.

Traffic Factor	Traffic Information
85th-percentile speed	49 mph (78.9 km/h)
50th-percentile speed	42 mph (67.6 km/h)
AADT	600 vehicles/d

Using the inputs from table 23, USLIMITS2 produced a recommended posted speed limit of 50 mph (80.5 km/h) for the SR 865 (Rockfish Road) study segment. The posted speed limit on SR 865 is 45 mph (72.4 km/h), which is less than the value recommended by the USLIMITS2 expert system.

Safety Performance of SR 865 (Rockfish Road)

Five yr (2011 through 2015) of crash data for SR 865 (Rockfish Road) were compiled to assess the historical safety performance of this roadway. On the study segment, there were a total of four crashes from the period, none of which was fatal. Table 31 shows the annual crash frequency distribution for the SR 865 case study segment.

Table 31. Historical crash data for the SR 865 (Rockfish Road) study segment.

Year	Total Crashes	Fatal and Injury Crashes
2011	0	0
2012	1	0
2013	2	1
2014	1	1
2015	0	0
Total	**4**	**2**

The crash prediction module of the IHSDM was utilized to predict crashes for the years 2011 through 2015. Crashes were predicted using the IHSDM crash prediction module with the EB

method and the HSM calibration factor. Those predicted numbers were compared to the reported crash data for the same 5-yr period. Table 32 compares the reported crashes to the predicted crashes.

Table 32. Comparison of reported to predicted crashes on SR 865.

Years	Total Crashes	Fatal and Injury Crashes
Reported crashes during 2011–2015	4	2
IHSDM crash prediction using EB method	3.43	1.36

As shown in table 25, the predicted number of crashes from the IHSDM is approximately the same as the number of reported crashes during the 5-yr analysis period. However, the number of predicted crashes is slightly less than the number of reported crashes. A total of four crashes were reported from 2011 through 2015, while the IHSDM with the EB adjustment predicted 3.43 crashes. This indicates that more crashes were reported than were expected on this case study segment.

Candidate Methods to Produce Consistency Between the Posted Speed Limit and Operating Speeds

Candidate methods to produce consistency between operating speeds and the posted speed limit include geometric changes to the roadway (if reconstruction is planned), or application of signs and pavement markings that can be applied to the existing roadway. Some of the signs and pavement markings currently on the SR 865 study segment include a posted speed limit sign, school warning pavement markings, and centerline and shoulder pavement markings. Other signs and pavement markings, in addition to the current signs and pavement markings, could be introduced in the SR 865 study segment that might possibly reduce speeds and produce operating speeds that are consistent with the posted speed limit.

Based on high-speed traffic calming treatments, various signs and pavement markings that could reduce speeds on tangent sections of the study segment include transverse pavement markings with speed feedback signs. In a study, this combination of sign and pavement markings has shown to have the potential to reduce 85th-percentile speeds by 4 mph (6.4 km/h). (Boodlal et al. 2015) Adding these treatments to the tangent sections could reduce speeds on the tangents and reduce speeds of motor vehicles traveling into the horizontal curves. The transverse pavement markings could create an illusion that drivers are operating at speeds faster than they actually are. This could then possibly reduce driving speeds. The speed feedback sign would inform drivers of their speed and especially draw attention to drivers who are traveling over the posted speed limit. If a speed feedback sign is added solely, without pavement markings, it has displayed the ability to reduce 85th-percentile speeds from 2 to 7 mph (3.2 to 11.3 km/h). (Boodlal et al. 2015)

In addition to the treatments mentioned previously that aim to reduce speeds on tangent sections of SR 865, other treatments could be added to reduce operating speeds on horizontal curves. Such treatments include curve warning signs in combination with chevron signs that span the horizontal curve. While no speed reduction effectiveness evaluation was performed, they pose the potential to reduce operating speeds on horizontal curves. (Boodlal et al. 2015) Adding these

signs would alert drivers to the approaching horizontal curve so they could reduce their speeds. Additionally, when an advisory speed limit sign was used for certain geometric conditions that constitute a lower speed, such as a sharp curve, a 15-percent reduction in 85th-percentile speeds was observed. (Boodlal et al. 2015)

The signs and pavement markings mentioned in this section have the potential to reduce speeds on the SR 865 study segment. The treatments vary between being suitable for application on tangent sections or on curve sections of the roadway. Each sign or pavement marking can be used on its own, or signs and pavement makings can be used in combinations to possibly produce operating speeds that are consistent with the posted speed limit.

CHAPTER 6. CONCLUSIONS

Speeding is cited as a contributory factor in nearly one-third of all fatal crashes reported in the United States. A significant number of these incidents occur on rural roadways with posted speed limits that exceed 40 mph. As a result, managing speeds on two-lane rural highways is likely to be an effective safety-management strategy. Application of self-enforcing, or self-explaining, roadways is one possible approach to manage speeds. In this context, a self-explaining or, self-enforcing, roadway is defined as one that encourages driver speed choice that is compliant with the regulatory speed limit.

This guidance report offers the following six conceptual approaches to design self-enforcing, or self-explaining, roadways:

1. Speed feedback loop in the geometric design process.
2. Inferred design speed approach.
3. Application of design consistency methods.
4. Application of existing geometric design criteria.
5. Use of signs and pavement markings.
6. Setting rational speed limits.

These methods can be applied individually or in combination for planned or existing two-lane rural highways. Example implementation methods are offered in this report, including two case studies of existing two-lane rural highways.

As the application of self-enforcing, or self-explaining, roadway design concepts becomes more commonplace in the United States and elsewhere, it is recommended that future research be undertaken to evaluate the effects of these practices on speed and safety. This report considers the relationship between various speed concepts and safety based on two case studies; however, these relationships should be explored across a variety of geographic locations to determine how speed concepts relate to safety performance on two-lane rural highways. Continuous, real-time speed information is available from data sources, such as the Second Strategic Highway Research Program (SHRP2) Naturalistic Driving Study and traffic monitoring systems, that can be used to link operating speeds to safety and geometric design features. The SHRP2 Roadway Information Database is another potential source for identifying geometric design features of roadways. Research using this information would be valuable in furthering the self-enforcing or self-explaining road concepts described in this guidance report. Lastly, driving simulators offer the opportunity to better understand how drivers select their speed in a controlled roadway environment. This affords the opportunity to study how combinations of pavement markings and signs, or altering geometric design criteria, may affect operating speeds.

APPENDIX A. OPERATING SPEED EQUATIONS

Table 33. Equations from literature used to determine operating speeds.

Study	Year	No.	Equation	R^2	Variables	Units	Conditions/Notes
Fitzpatrick et al.	2005	1	$* FF_{85} = 12.4 + 0.98(SL)$	0.904	SL	Metric	—
Fitzpatrick et al.	2005	2	$* FF_{85} = 25.9 + 0.83(SL) - 0.054(AD)$	0.923	SL, AD	Metric	For sites with posted speed limit of 45 mph (73 km/h) or less
Fitzpatrick et al.	2005	3	$* FF_{85} = 65.0 - 12.2\ park$	0.50	park	Metric	For sites with 30 mph (48 km/h) posted speed limit
Misaghi and Hassan	2005	1	$\dagger V_{85MC} = 91.85 + 9.81 \times 10^{-3} R$	0.464	R	Metric	—
Misaghi and Hassan	2005	2	$\dagger V_{85MC} = 94.30 + 8.67 \times 10^{-6} R^2$	0.524	R	Metric	—
Misaghi and Hassan	2005	3	$\dagger \triangle_{85}V = -83.63 + 0.93V_T + e^{-8.93+3507.10/R}$	0.649	R, V_T	Metric	—
Misaghi and Hassan	2005	4	$\dagger \triangle_{85}V = -194.74 + 21.42\left(\sqrt{V_T}\right) + 0.11DFC - 4.55SW - 5.36(curve - dir) + 1.30G + 4.22(drv_{flag})$	0.889	V_T, DFC, SW, curve direction, G, drv_{flag}	Metric	—
Fitzpatrick et al.	2003	1	$EV_{85} = 7.675 + 0.98(PSL)$	0.904	PSL	U.S. customary	All roadways
Fitzpatrick et al.	2003	2	$EV_{85} = 8.666 + 0.963(PSL)$	0.86	PSL	U.S. customary	Suburban/urban arterial
Fitzpatrick et al.	2003	3	$EV_{85} = 21.131 + 0.639(PSL)$	0.41	PSL	U.S. customary	Suburban/urban collector
Fitzpatrick et al.	2003	4	$EV_{85} = 10.315 + 0.776(PSL)$	0.14	PSL	U.S. customary	Suburban/urban local
Fitzpatrick et al.	2003	5	$EV_{85} = 36.453 + 0.517(PSL)$	0.81	PSL	U.S. customary	Rural arterial
Schurr et al.	2002	1	$\ddagger V_{85,midpoint} = 103.3 - 0.1253\Delta + 0.0238LC + 1.039G_1$	0.46	Δ, LC, G_1	Metric	—
Schurr et al.	2002	2	$\ddagger V_{85,approach} = 70.2 + 0.434V_p - 0.001307T_{ADT}$	0.19	V_p, T_{ADT}	Metric	—

Study	Year	No.	Equation	R^2	Variables	Units	Conditions/Notes
Schurr et al.	2002	3	$\ddagger V_{95,midpoint} = 113.9 - 0.122\Delta + 0.0178LC - 0.00184T_{ADT}$	0.41	Δ, LC, T_{ADT}	Metric	—
Schurr et al.	2002	4	$\ddagger V_{95,approach} = 84.4 + 0.352V_p - 0.001399T_{ADT}$	0.22	V_p, T_{ADT}	Metric	—
Fitzpatrick et al.	2000a	1	$V_{85} = 106.30 - \dfrac{3595.29}{R}$	0.92	R	Metric	Horizontal curve on grade: $0 \le G < 4$ or horizontal curve combined with sag vertical curve
Fitzpatrick et al.	2000a	2	$V_{85} = 96.46 - \dfrac{2744.49}{R}$	0.56	R	Metric	Horizontal curve on grade: $4 \le G < 9$
Fitzpatrick et al.	2000a	3	$V_{85} = 100.87 - \dfrac{2720.78}{R}$	0.59	R	Metric	Horizontal curve on grade: $-9 \le G < 0$
Fitzpatrick et al.	2000a	4	$V_{85} = 101.90 - \dfrac{3283.01}{R}$	0.78	R	Metric	Horizontal curve combined with limited sight distance crest vertical curve
Fitzpatrick et al.	2000a	5	$V_{85} = 111.07 - \dfrac{175.98}{K}$	0.54	K	Metric	Vertical crest curve with limited sight distance on horizontal tangents
Fitzpatrick et al.	2000a	6	$V_{85} = 100.19 - \dfrac{126.07}{K}$	0.68	K	Metric	Sag vertical curve on horizontal tangent
Polus et al.	2000	1	$\ddagger SP = 102.5 - 37.34/e^{(0.00668 \times GM_L)}$	0.332	GM_L	Metric	$GM_L \le 200$; maximum speed = 63.69 mph (102.5 km/h)
Polus et al.	2000	2	$\ddagger SP = 105.00 - 21.30/e^{(0.00092 \times GM_L)}$	0.228	GM_L	Metric	$GM_L \le 1000$; maximum speed = 65.25 mph (105 km/h)
Polus et al.	2000	3	$\ddagger SP = 101.11 - 3420/GM_S$	0.553	GM_S	Metric	Small radii and small TL; $R \le 250$ m; TL < 150 m
Polus et al.	2000	4	$\ddagger SP = 98.405 - 3184/GM_L$	0.684	GM_L	Metric	Small radii and intermediate TL; $R \le 250$ m; 150 m < TL < 1000 m; $GM_L \le 1500$
Polus et al.	2000	5	$\ddagger SP = 105.00 - 28.107/e^{(0.00108 \times GM_L)}$	0.742	GM_L	Metric	If maximum 85th-percentile speed is 65.25 mph (105 km/h)

Study	Year	No.	Equation	R^2	Variables	Units	Conditions/Notes
Polus et al.	2000	6	$\ddagger SP = 105.00 - 22.953/e^{(0.00012 \times GM_L)}$	0.838	GM_L	Metric	Large TL and any reasonable radius; $TL < 1000$ m
Krammes et al.	1995	1	$V_{85} = 103.66 - 1.95D$	0.80–0.82	D	Metric	85th-percentile speed on long tangents overestimated
Krammes et al.	1995	2	$V_{85} = 102.45 - 1.57D + 0.0037LC - 0.10I$	0.82	D, LC, I	Metric	—
Krammes et al.	1995	3	$V_{85} = 41.62 - 1.29D + 0.0049LC - 0.12I + 0.95V_T$	0.90	D, LC, I, V_T	Metric	—
Lamm and Choueiri	1987	1	$\ddagger V_{85} = 34.700 - 1.00(DC) + 2.081(LW) + 0.174(SW) + 0.0004(AADT)$ [mph]	0.842	$DC, LW, SW, AADT$	U.S. customary	—
Lamm and Choueiri	1987	2	$\ddagger V_{85} = 58.656 - 1.135(DC)$ [mph]	0.787	DC	U.S. customary	—
Lamm and Choueiri	1987	3	$\ddagger V_{85} = 55.132 - 0.042(CCR)$ [mph]	0.846	CCR	U.S. customary	Lane width = 10 ft
Lamm and Choueiri	1987	4	$\ddagger V_{85} = 55.646 - 1.019(DC)$ [mph]	0.753	DC	U.S. customary	Lane width = 10 ft
Lamm and Choueiri	1987	5	$\ddagger V_{85} = 57.602 - 0.040(CCR)$	0.731	CCR	U.S. customary	Lane width = 11 ft
Lamm and Choueiri	1987	6	$\ddagger V_{85} = 58.310 - 1.052(DC)$	0.746	DC	U.S. customary	Lane width = 11 ft
Lamm and Choueiri	1987	7	$\ddagger V_{85} = 59.515 - 0.038(CCR)$	0.836	CCR	U.S. customary	Lane width = 12 ft
Lamm and Choueiri	1987	8	$\ddagger V_{85} = 59.746 - 0.998(DC)$	0.824	DC	U.S. customary	Lane width = 12 ft
McLean	1979	1	$\ddagger V_{85} = 53.8 + 0.464V_F - 3.26\left(\frac{1}{R}\right) \times 10^3 + 8.5\left(\frac{1}{R}\right)^2 \times 10^4$	0.92	V_F, R	Metric	—
Fitzpatrick et al.	1995	1	$V_{85,tan} = 74.91 + \dfrac{22.29}{AD}$	0.71	AD	Metric	Suburban highways, tangent sections
Fitzpatrick et al.	1995	2	$V_{85,curve} = 54.18 + 1.061R^{0.5}$	0.72	R	Metric	Suburban highways, curve sections
Fitzpatrick et al.	1995	3	$V_{85,curve} = 56.34 + 0.808R^{0.5} + 9.34/AD$	0.81	R, AD	Metric	Suburban highways, curve sections
Banihashemi et al.	2011	1	$\ddagger V_{T85} = 35.15 + 0.26PS - \dfrac{1132}{R_P}$	0.49	R_P	U.S. customary	Tangents with $L_T < 150$ ft
Banihashemi et al.	2011	2	$\ddagger V_{T85} = 26.04 + 0.53PS - 0.89RHR + 0.005L_T$	0.29	PS, RHR, L_T	U.S. customary	Tangents with $L_T \geq 150$ ft

Study	Year	No.	Equation	R^2	Variables	Units	Conditions/Notes
Banihashemi et al.	2011	3	$\ddagger V_{C85} = 44.25 - \dfrac{1462}{R}$	0.37	R	U.S. customary	Curve speed

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—No information.

FF_{85} = 85th-percentile free-flow operating speed (mph (km/h)); SL = posted speed limit (mph (km/h)); AD = access density; *park* = indicator variable for the presence of parking; V_{85MC} = 85th-percentile speed at middle of curve (mph (km/h)); $\Delta_{85}V$ = 85th-percentile speed differential calculated as 85th-percentile value of speed differentials of individual drivers; V_T = approach tangent speed (mph (km/h)); DFC = deflection angle of circular curve (degrees); SW = shoulder width (ft (m)); *curve – dir* = curve direction; drv_{flag} = driveway flag; EV_{85} = estimated 85th-percentile speed (mph (km/h)); PSL = posted speed limit (mph (km/h)); $V_{85, midpoint}$ = 85th-percentile speed at free-flow passenger cars at the curve midpoint (mph (km/h)); Δ = deflection or intersection angle of horizontal curve (degrees); LC = length of horizontal curve (ft (m)); G_1 = approach grade (percent); $V_{85, approach}$ = 85th-percentile speed at the approach location (mph (km/h)); V_p = posted speed limit (mph (km/h)); T_{ADT} = average daily traffic (vehicles per day); $V_{95, midpoint}$ = 95th-percentile operating speed at the midpoint of the horizontal curve (mph (km/h)); $V_{95, approach}$ = 95th-percentile operating speed on the approach tangent to the horizontal curve (mph (km/h)); K = rate of vertical curvature (ft/percent (m/percent)); SP = posted speed limit (mph (km/h)); GM_L = geometric measure for long tangents between horizontal curves (ft^2 (m^2)); GM_S = geometric measure for short tangents between horizontal curves (ft^2 (m^2)); D = degree of curvature; I = deflection or intersection angle of horizontal curve (degrees); LW = lane width (ft (m)); CCR = curvature change rate (degrees per half mile); V_F = desired speed of the 85th-percentile car (mph (km/h)); $V_{85, tan}$ = estimated 85th-percentile operating speed on tangent (mph (km/h)); $V_{85, curve}$ = 85th-percentile speed on the curve (mph (km/h)); V_{T85} = estimated 85th-percentile operating speed on tangent (mph (km/h)); PS = posted speed limit (mph (km/h)); R_p = radius of preceding horizontal curve (ft (m)); RHR = roadside hazard rating; L_T = length of approach tangent (ft (m)); V_{C85} = estimated 85th-percentile operating speed on the curve (mph (km/h)).

APPENDIX B. INFERRED DESIGN SPEED CALCULATIONS USING HORIZONTAL AND VERTICAL CURVATURE

This appendix shows sample calculations used to determine the inferred design speed of a roadway segment based on geometric design data. The given information shown below is typical of data found on roadway design plans or data that can be measured in the field. The sample calculations include equations for SSD, length of vertical curvature, horizontal alignment design, and HSO.

Given:
  Radius of curve = 955 ft (291.1 m).
  Rate of superelevation rate = 6.1 percent.
  Available SSD = 528 ft (160.9 m).
  Sag vertical curve.
  G_1 = –1.5 percent.
  G_2 = –0.5 percent.
  L = 200 ft (61.0 m).
  HSO = 20 ft (6.1 m).

SSD

The SSD equation found in the Green Book (equation 3-2 in the 2011 Green Book) can be used to solve for the inferred design speed. (AASHTO 2011) For a level roadway, figure 69 is used.

$$SSD = 1.47Vt + 1.075\frac{V^2}{a}$$

Figure 69. Equation. Available SSD.

Where:
  SSD = available stopping sight distance (ft (m)).
  V = inferred design speed (mph (km/h)).
  t = perception–reaction time (2.5 s).
  a = deceleration rate (11.2 ft/s^2 (3.4 m/s^2)).

Substituting the given information to the SSD equation becomes figure 70.

$$528 = (1.47)V(2.5) + 1.075\frac{V^2}{11.2}$$

Figure 70. Equation. Using available SSD equation to solve for inferred design speed.

Solving for inferred design speed using the quadratic equation, the inferred design speed is V = 57.46 mph (92.5 km/h).

LENGTH OF VERTICAL CURVE

The following vertical alignment design equations (figure 71 though figure 74) can be used to calculate inferred design speed based on the length of vertical curve.

$$A = |G_2 - G_1|$$

Figure 71. Equation. Algebraic difference in grades.

$$A = |(-0.5) - (-1.5)| = 1$$

Figure 72. Equation. Inputting given information to solve for algebraic difference in grades.

$$L = KA$$

Figure 73. Equation. Length of vertical curve.

$$K = \frac{200}{1} = 200$$

Figure 74. Equation. Inputting given information to solve for length of vertical curve.

Where K is the rate of vertical curvature (ft/percent difference in grades (m/percent difference in grades)).

Table 3-36 in the Green Book can be used to determine the minimum SSD and design speed for a given rate of vertical curvature. (AASHTO 2011) Using a rate of vertical curvature of 200 and interpolating the design speed data shown in table 3-36, the following values are determined: SSD = 800.08 ft (243.9 m) and design speed = 73.89 mph (118.9 km/h).

RADIUS OF CURVATURE AND SUPERELEVATION

The inferred design speed can be calculated for combinations of horizontal curvature and superelevation. The following equation (figure 75) from the Green Book (equation 3-9 in the Green Book) is referred to as the point–mass model, and it can be used to determine the inferred design speed based on the radius of curve–superelevation combination.

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

Figure 75. Equation. Point–mass model to determine inferred design speed. (AASHTO 2011)

Both the speed and side-friction demand factor are unknown. Rearranging the point–mass model to solve for the side-friction demand factor produces the following equations (figure 76 through figure 78):

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

Figure 76. Equation. Point–mass model to solve for the side-friction demand factor.

$$f = \frac{V^2}{15(955)} - 0.01(6.1)$$

Figure 77. Equation. Inputting given information into the point–mass model.

$$f = \frac{V^2}{14325} - 0.061$$

Figure 78. Equation. Reducing inputted information in the point–mass model.

A trial-and-error process, using various design speeds and the information shown in table 3-7 from the Green Book, can be used to solve for the side-friction demand factor that does not exceed the maximum side-friction demand factor for a given design speed. (AASHTO 2011) Several iterations of this process will likely occur to obtain the inferred design speed.

First, a design speed of 50 mph (80.5 km/h) was used with the given superelevation of 6.1 percent. From table 3-7 in the Green Book, a design speed of 50 mph (80.5 km/h) has a maximum friction value of 0.14. The values were inputted into the equation (figure 79) to determine if the calculated friction value exceeded the given value.

$$f = \frac{50^2}{14325} - 0.061 = 0.11352$$

Figure 79. Equation. Calculating the side-friction demand factor for a design speed of 50 mph (80.5 km/h).

The f value of 0.114 is smaller than the maximum f value of 0.14; therefore, a larger inferred design speed is considered.

The second iteration applied a design speed of 55 mph (88.5 km/h) with the given superelevation of 6.1 percent. From table 3-7 in the Green Book, the maximum friction factor for an inferred design speed of 55 mph (88.5 km/h) is 0.13 (figure 80).

$$f = \frac{55^2}{14325} - 0.061 = 0.150$$

Figure 80. Equation. Calculating the side-friction demand factor for a design speed of 55 mph (88.5 km/h).

The f value of 0.150 exceeds the maximum friction value of 0.13; therefore, a smaller inferred design speed should be considered.

A third iteration was completed using a design speed of 53 mph (85.3 km/h) with the given superelevation of 6.1 percent (figure 81). Interpolating from table 3-7 in the Green Book produces a maximum friction factor of 0.134 for an inferred design speed of 53 mph (85.3 km/h).

$$f = \frac{53^2}{14325} - 0.061 = 0.135$$

Figure 81. Equation. Calculating the side-friction demand factor for a design speed of 53 mph (85.3 km/h).

The f value of 0.135 exceeds the maximum friction value of 0.134; therefore, a smaller inferred design speed should be considered.

A fourth iteration was completed using a design speed of 52 mph (83.7 km/h) with the given superelevation of 6.1 percent (figure 82). Interpolation from table 3-7 in the Green Book produces a maximum side friction factor of 0.136 for an inferred design speed of 52 mph (83.7 km/h).

$$f = \frac{52^2}{14325} - 0.061 = 0.128$$

Figure 82. Equation. Calculating the side-friction demand factor for a design speed of 52 mph (83.7 km/h).

The f value of 0.128 does not exceed maximum friction value of 0.136; therefore, the inferred design speed for the horizontal curve is 52 mph (83.7 km/h).

HSO

The equation for the HSO can be used to calculate the radius of curvature, which can then be substituted into the horizontal curve equation to determine an inferred design speed. The following equation (figure 83) from the Green Book (equation 3-36 in the Green Book) is used for this purpose.

$$HSO = R\left[1 - \cos\left(\frac{28.65S}{R}\right)\right]$$

Figure 83. Equation. HSO. (AASHTO 2011)

Where:
  S = stopping sight distance (ft (m)).
  R = radius of curve (ft (m)).

Substituting the given information produces figure 84.

$$20 = R\left[1 - \cos\left(\frac{28.65(528)}{R}\right)\right]$$

Figure 84. Equation. Using HSO equation to solve for radius of curve.

Where R is 1739.31 ft (530.1 m).

Using the horizontal curve equation for figure 85 through figure 87 as follows:

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

Figure 85. Equation. Point–mass model to determine inferred design speed.

$$f = \frac{V^2}{15(1739.31)} - 0.01(6.1)$$

Figure 86. Equation. Inputting given information into the point–mass model (step 1).

$$f = \frac{V^2}{26089.65} - 0.061$$

Figure 87. Equation. Inputting given information into the point–mass model (step 2).

First, a trial design speed of 70 mph was used with a given superelevation of 6.1 percent. From table 3-7 in the Green Book, the maximum friction for a design speed of 70 mph (112.7 km/h) is 0.10 (figure 88).

$$f = \frac{70^2}{26089.65} - 0.061 = 0.127$$

Figure 88. Equation. Calculating the side-friction demand factor for a design speed of 70 mph (112.7 km/h). (AASHTO 2011)

The f value of 0.127 exceeds the maximum friction value of 0.10; therefore, a lower inferred design speed is considered.

The second iteration considered a design speed of 67 mph (107.8 km/h) with the given superelevation of 6.1 percent. From table 3-7 in the Green Book, the maximum friction value for a design speed of 67 mph (107.8 km/h) is 0.106 (figure 89).

$$f = \frac{67^2}{26089.65} - 0.061 = 0.111$$

Figure 89. Equation. Calculating the side-friction demand factor for a design speed of 67 mph (107.8 km/h). (AASHTO 2011)

The f value of 0.111 exceeds the maximum friction value of 0.106; therefore, a lower inferred design speed is considered.

The third iteration used a design speed of 66 mph with the given superelevation of 6.1 percent. From table 3-7 in the Green Book, the maximum friction value for a design speed of 66 mph (106.2 km/h) is 0.108 (figure 90).

$$f = \frac{66^2}{26089.65} - 0.061 = 0.106$$

Figure 90. Equation. Calculating the side-friction demand factor for a design speed of 66 mph (106.2 km/h). (AASHTO 2011)

The f value of 0.106 does not exceed the maximum friction value of 0.108; therefore, the inferred design speed for the horizontal curve using the HSO is 66 mph (106.2 km/h).

CONCLUSION

The inferred design speed for this example site, which includes an overlapping horizontal and vertical curve, is the lowest among the values computed for each speed-influencing factor. As such, the horizontal curve radius–superelevation combination, with an inferred design speed of 52 mph (83.7 km/h), is the controlling geometric design feature.

GLOSSARY

85th-percentile speed	Speed at which 85 percent of the motor vehicles are traveling at or below.
Braking distance (d_b)	Distance traveled by a vehicle once the brakes are applied until the vehicle completely stops (ft (m)).
Designated design speed (design speed)	Speed used to establish certain geometric elements of a roadway (mph (km/h)).
Desired speed	Speed drivers wish to travel based on the alignment characteristics of a roadway, including topography, cross section, adjacent land use, and traffic volumes.
Green Book	*A Policy on the Geometric Design of Highways and Streets*. (AASHTO 2011)
Horizontal sight line offset (HSO)	Lateral offset along a horizontal curve measured from the centerline of the innermost travel lane to a sight line obstruction (ft (m)).
Inferred design speed	Speed at which all critical geometric design-related criteria are met at a certain point along a roadway (mph (km/h)).
Lane width	Portion of roadway designated for motor vehicle use (ft (m)).
Length of vertical curvature (L)	Distance from the point of vertical curve (PVC) to the point of vertical tangency (PVT) (ft (m)).
Maximum side friction factor (f_{max})	Maximum side friction demand used in horizontal curve design. The maximum side friction factor is based on driver comfort levels (i.e., tolerance for lateral acceleration) and is also referred to as the limiting side friction factor. (Torbic et al. 2014)
Maximum superelevation (e_{max})	Maximum cross slope of the roadway cross section within the limits of a horizontal curve. This value ranges from 4 to 12 percent, depending on climatic conditions, area type, terrain, and the frequency of very slow-moving vehicles in the traffic stream.
Operating speed	Speed at which motor vehicles are observed traveling on a given roadway during free-flow conditions (mph (km/h)).
Posted speed limit	Maximum speed that can be legally driven on a given roadway. The speed limit is typically posted on regulatory signs (mph (km/h)).

Radius of curvature (R)	Provides for transition between two tangent segments of roadway (ft (m)).
Rate of vertical curvature (K)	Horizontal distance necessary to affect a 1-percent change in the vertical gradient of the roadway, which is defined by the length of vertical curvature (L) divided by the algebraic difference in grades (A).
Shoulder width	Width of the area parallel to the travel lane and part of the roadway; the shoulder can be either paved or unpaved (ft (m)).
Stopping sight distance (SSD)	Distance required for a driver to perceive, react, and brake to a stop (ft (m)).
Target speed	Anticipated or desired operating speed of a roadway or section of roadway.
USLIMITS2	Web-based tool developed by the FHWA used for guidance regarding the setting of appropriate posted speed limits for all road types. (FHWA 2016b)

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