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SPEED HARMONIZATION—DESIGN SPEED VS. OPERATING SPEED

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16. Abstract

When the actual operating speed on the roads exceeds the design speed, which is common on rural highways, the roadway design may become problematic from a safety point of view. This report presents a new methodology that summarizes the relationship between design speed and operating speed, as well as the safety impacts of various geometric elements. A comprehensive literature review and a series of interviews with Illinois county engineers were conducted to summarize the current roadway design and maintenance practices and their impacts on Illinois roadway safety. An integrated modeling framework that includes modules for (1) geometric design simulation, (2) operating speed-profile prediction, and (3) crash rate prediction, is proposed. Based on this research, a benefit–cost analysis was also developed to quantify the economic benefits of various strategies for roadway safety improvement. All models were programmed into an Excel VBA-based computer tool to facilitate decision making. The outcome of this project may be suitable for implementation in a wide range of application contexts.

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EXECUTIVE SUMMARY

Current geometric design of roadway elements is based primarily on the design speed, which may be unrelated to the posted speed limit and/or the actual operating speed. When the actual operating speed on the roads exceeds the design speed, which is common on rural highways, the roadway design may become problematic from a safety point of view.

To quantify the relationship between design speed, operating speed, and roadway safety, this report presents a new methodology that summarizes the relationship between design speed and operating speed, as well as the safety impacts of various geometric elements. To understand current field practice and possible feasible strategies for speed harmonization, a comprehensive literature review and a series of interviews with Illinois county engineers were conducted; the roadway design and maintenance practices and their impacts on Illinois roadway safety are summarized in this report. An integrated modeling framework that includes modules for (1) geometric design simulation, (2) operating speed-profile prediction, and (3) crash rate prediction, is proposed. Based on this research, a new methodological framework for a benefit–cost analysis was also developed to quantify the economic benefits of various strategies for roadway safety improvement.

Numerical examples are presented to demonstrate the applicability of the proposed models. All models were programmed into an Excel VBA-based computer tool to facilitate decision making. The outcome of this project may be suitable for implementation in a wide range of application contexts.

CONTENTS

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Geometric design refers to the selection of roadway elements such as the horizontal alignment, vertical alignment, cross section, and roadside of a highway or street. A good geometric design must provide appropriate levels of mobility and land use access, while at the same time maintain a high degree of safety. The balance between mobility/accessibility and safety is often reflected by the "allowed" vehicle speed on the roadway element.

The design speed is usually used to determine geometric features of a new road during road design. In the 1994 edition of the AASHTO *Green Book*, the design speed was defined to be "the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern." The assumed design speed should be a logical one with respect to the topography, the adjacent land use, and the functional classification of highway. In 2004, the definition was changed to "a selected speed used to determine the various geometric design features of a roadway." Hence, design speed is critical for the design of a roadway, especially in choosing super-elevation rates and radii of curves, sight distance, and the lengths of crest and sag vertical curves. Roads with higher travel speeds require sweeping curves, steeper curve banking, longer sight distances, and smoother hill crests and valleys.

The operating speed of a road is the speed at which vehicles generally operate on that road. In the same 1994 edition of the AASHTO *Green Book*, the operating speed was defined as "the highest overall speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions without at any time exceeding the safe speed as determined by the design speed on a section-by-section basis." In July 2001, however, AASHTO revised their definition in the new edition of the *Green Book* and defined it as "the speed at which drivers are observed operating their vehicles during free-flow conditions." The 85th percentile of observed speeds is the most frequently used measure of operating speed.

Currently, the choice of design speed value in many states is often based on traffic volume and roadway functional classification, and this may cause a discrepancy with the actual operating speed of that same roadway. For example, a very low-volume road functionally classified as local may have a design speed of 30 mph. However, the regulatory posted speed may be 55 mph, and the operating speed may be even higher. The ranges of such deviation are partially reflected in the design-class flow chart in Figure 1, where the shown speed is generally the anticipated operating speed—often used as design speed. (Note, however, that the actual operating speed is often higher than the anticipated operating speed.)

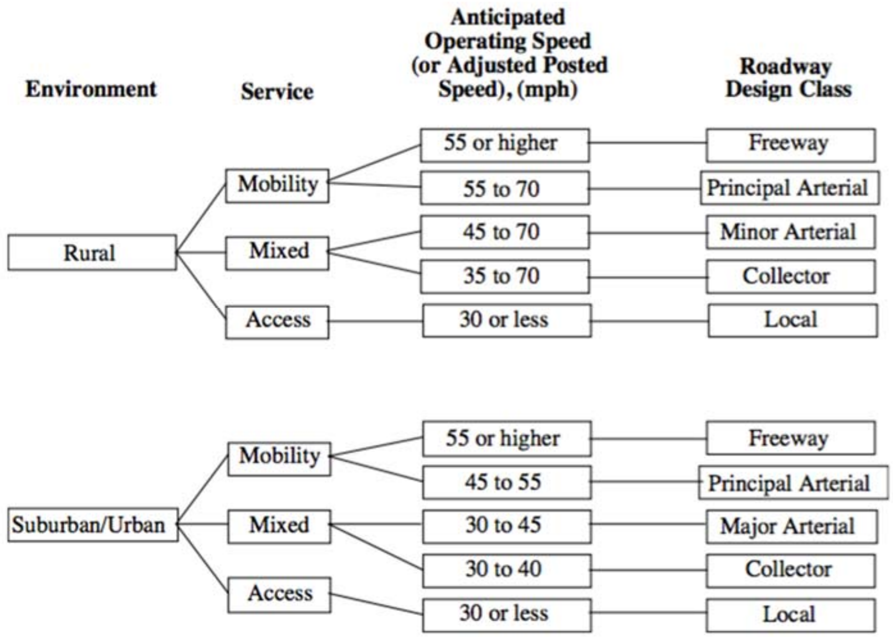


Figure 1: Roadway design-class flow chart (Fitzpatrick et al. 2003)

The discrepancy between design speed and operating speed is potentially problematic from a safety point of view. Safety features of geometric design (such as clear zone and guardrail length) are determined based on the lower design speed rather than the actual operating speed. Of particular concern are locations with a sheer drop-off, such as an 8 ft deep box culvert just outside the policy clear zone. The clear zone for local roads can be as low as 6 ft for roads with average daily traffic (ADT) ≤ 400, whereas 55 mph design speed on a 750 ADT road with relatively flat front slopes may require a clear zone of 16 ft. Is it reasonable to accept a clear zone based on design speed, or is a higher design based on operating speed more appropriate?

1.2 PROBLEM STATEMENT AND PROJECT SCOPE

There is a question about whether legal speed or operating speed is more appropriate than design speed when designing clear zones and safety features. This is especially true for low-volume roads with high operating speeds. Are coefficients used in safety design for higher ADT roadways acceptable, or should they be calibrated for lower-volume roads? Should a researched and risk-based consideration of safety features based on design speed vs. operating speed be implemented? At this time, there are locations such as a deep culvert just outside a clear zone (as noted previously) that may not meet policy for a higher design speed. Some engineers find the clear zone policy acceptable for the design speed on low-volume local roads, but others are uncomfortable with the policy when the operating speed is higher, based on engineering judgment.

This study aims to quantify the impact of design speed vs. legal/posted speed and operating speed on highway design safety. The results will help the Illinois Department of Transportation (IDOT) better assess the impact of design speed choices, improve its design practice, and ultimately improve safety without compromising mobility and accessibility. More specifically, this study will answer the question of whether legal speed or operating speed is more appropriate than design speed when designing safety features. We will have a better understanding on whether the coefficients used in safety design for higher ADT roadways are acceptable for lower-volume roads. We will also develop a new risk-based analysis framework for safety features based on design speed vs. operating speed. Also, a benefit–cost analysis will be conducted.

The rest of the report is organized as follows: Section 2 provides a comprehensive review of the literature that links design speed, geometric design features, operating speed, and safety impacts. Section 3 describes the methodology we propose to quantitatively analyze the safety impacts of speed harmonization. Section 4 shows an illustrative example to demonstrate our framework. Section 5 discusses directions for future research.

CHAPTER 2: LITERATURE REVIEW

2.1 VARIOUS SPEEDS

Highway safety, a vital aspect of the modern transportation system, is closely related to the speed of vehicles. As suggested by "A Policy on Geometric Design of Highways and Streets" (AASHTO 2001), the design speed is selected to determine the roadway geometric features and help ensure safety on the highway.

Speed limit is posted to inform motorists of the highest speed considered to be safe and reasonable under favorable road, traffic, and weather conditions. The posted speed limit usually sets the maximum speed limit for a roadway such that the operating speed may be above the design speed for a particular location of the roadway. The 85th percentile speed, defined as the speed at or below which 85% of the traffic is traveling, is commonly used by highway agencies for describing actual operating speeds and establishing speed limits.

However, the design approach based on design speed may encounter safety issues due to inconsistencies among design speed, operating speed, and the posted speed limit. According to Fitzpatrick et al. (2003), it was generally preferable to establish a strong relationship among design speed, operating speed, and posted speed limited, which indicates that those criteria should have similar values. Fitzpatrick et al. (2003) illustrated the relationship between operating speed and the posted speed limit as a relatively linear relationship (such as shown in Figure 2) based on analysis of data from different sites. According to that research, operating speeds could be greater than posted speed limits on most types of roadways. Usually, operating speeds were 5 to 15 mph higher than the posted speed limits, while operating speeds in some rural areas could be lower than the posted speed limits. The relationship varies between sites, e.g., the difference between operating speed and speed limit tends to be very large in the rural arterial roads with a posted speed limit of 45 mph. While for the same posted speed limit, the difference between these two speeds was very small in the suburban/urban arterial roads. Also, a survey conducted by Schroeder et al. (2013) showed that 9.8% of the interviewees drove often or at least sometimes 15 mph over the speed limit on two-lane highways. And 19.1% of the participants admitted to driving 15 mph higher than the posted speed limits on multi-lane highways.

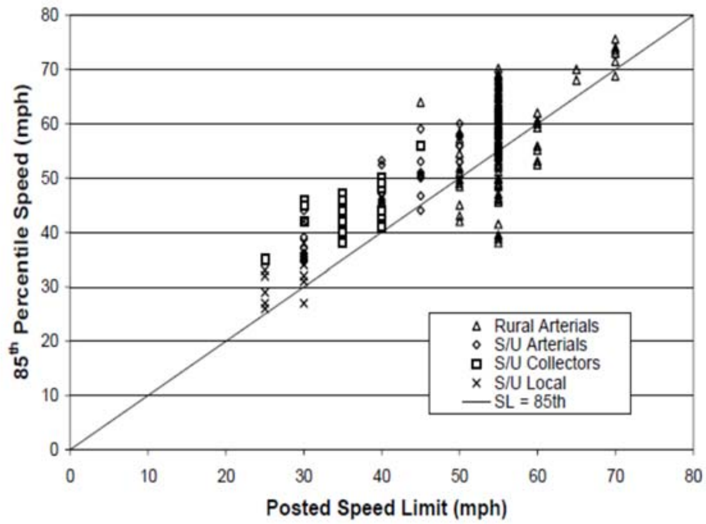


Figure 2: The empirical relationship between operating speed and posted speed limit (Fitzpatrick et al. 2003).

A study by Fitzpatrick et al. (2003) noted that the operating speeds were generally higher than the design speeds on the rural two-lane highway (Figure 3a). The relationship between design speeds and operating speeds was also found to be unclear or even irrelevant. The results shown in Figure 3 would obviously lead to the conclusion that the operating speeds vary in almost the same range regardless of the change in design speed. For example, in the rural two-lane highway, even the design speed increases from 30 to 60 km/h, the change in the actual operating speed was still insignificant, which means the current design fails to build a reliable relationship between these two speeds on the rural two-lane highway. However, in suburban arterial roads, the relationship between these two speeds is quite different. The operating speed might be smaller than the design speed, which could be explained by the congested traffic condition.

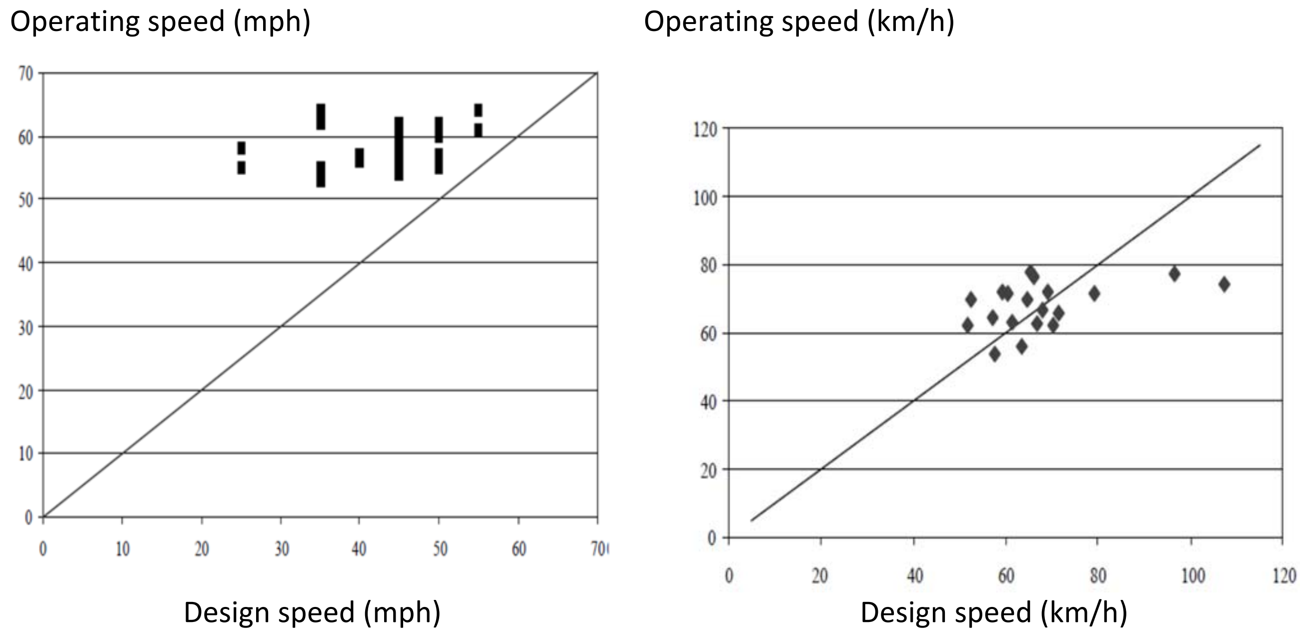


Figure 3: Design speed versus observed operating speed: (a) rural two-lane highway; (b) suburban arterial (Fitzpatrick et al. 2003).

2.2 DESIGN SPEED PRACTICE

Driving speeds are influenced by geometric elements such as horizontal curvature, vertical curvature, and cross-section. Designing the geometric elements based on speeds not harmonized with actual speeds can result in inappropriate road elements that are less than optimal for safety and comfort.

The current geometric design policies are based on the design speed concept. A road design procedure begins with selecting a design speed that is used for determination of the various geometric design elements of the roadway.

The main factors that are used most frequently when selecting a design speed for a new road in the United States are (from high to low): (1) functional classification, (2) legal speed limit (legislative or maximum value), (3) legal speed limit plus 5 or 10 mph (8.1 to 16.1 km/h), (4) anticipated traffic volume, and (5) anticipated operating speed according to a survey conducted by Fitzpatrick et al. (2003). Among all the factors, the functional classification is mostly used in different design methods (e.g., AASHTO policy, state DOT survey). In many of the state DOT design methods, the legal speed limit, terrain type, and traffic volume are also taken into consideration when selecting the design speed. In general, higher design speed can be chosen for areas with flatter terrain and higher ADT. The factors used to select design speed are summarized in Table 1.

Table 1: Factors Used to Select Design Speed (Fitzpatrick et al. 2003)

AASHTO Policy	State DOT Survey	International Practices
Functional classification Rural versus urban Terrain type	Functional classification Legal speed limit Legal speed limit plus a value (e.g., 5 or 10 mph [8.1 to 16.1 km/h]) Anticipated volume Anticipated operating speed Terrain type Development Costs Consistency	Anticipated operating speed Feedback loop

At IDOT, the functional classification system is used as the foundation for highway planning and geometric design (*Bureau of Local Roads and Streets Manual* 2005). The system considers five classes: (1) principal arterials, (2) minor arterials, (3) major collectors, (4) minor collectors, and (5) local. The types of development areas (rural highway and roads, urban roads and streets) where the project is located further divides the criteria. The selection of design speed is based primarily on this classification, with minor consideration given to operating speed, as is the case in several design methods used in the United States.

However, the design speed selected through the AASHTO procedure might fail to provide a sufficiently safe design. According to Krammes (2000), the design speeds for arterial roads in rolling terrain and the collectors in level and rolling terrain were smaller than the driver's desired speeds. Based on the smaller design speed, smaller stopping sight distances could be used in reality, and it would be more difficult for drivers to take actions to avoid the collision while driving at such high speeds (Wilmot et al. 1999). Thus, the current design may underestimate the risk.

Krammes (2000) also thought that AASHTO should determine the appropriate design speed based on new data because the conditions of roads and vehicles have changed during the past few decades. Also, he suggested that a percentile operating speed higher than the 85th could be considered if the revision for the current design was needed. Moreover, he stated that AASHTO's procedure for determining maximum side-friction factors and super-elevation rates should be modified. Especially for the procedure of determining super-elevation rates, he thought the design speed should be replaced by the operating speed because the super-elevation rates can be felt by drivers only after they enter a curve.

2.3 SPEED STATUTES

2.3.1 Illinois

In Illinois, the statutory speed limit is set for different districts and the vehicle types. The speed limits for each category is summarized in Table 2.

Table 2: Illinois Statutory Speed Limits (Policy on Establishing and Posting Speed Limits on the State Highway System 2011)

<table>
<tr><th>District</th><th>Roadway Type</th><th colspan="2">Vehicle Type</th><th>Speed Limit</th></tr>
<tr><td rowspan="6">Outside Urban Districts</td><td rowspan="5">Freeways/Expressways</td><td colspan="2">Passenger cars, buses, and trucks with gross weights of 4 tons or less</td><td>65 mph</td></tr>
<tr><td colspan="2">Vehicles towing trailers, house cars, and campers</td><td>65 mph</td></tr>
<tr><td rowspan="3">Trucks with gross weights of over 4 tons</td><td>Interstate Routes</td><td>65 mph</td></tr>
<tr><td>All Non-Interstate Routes</td><td>55 mph</td></tr>
<tr><td>Within Cook, DuPage, Kane, Lake, McHenry, and Will Counties</td><td>55 mph</td></tr>
<tr><td>Conventional highways</td><td colspan="2">All vehicle types</td><td>55 mph</td></tr>
<tr><td rowspan="2">Inside Urban Districts</td><td>All streets and highways</td><td colspan="2">All vehicle types</td><td>30 mph</td></tr>
<tr><td>Alleys</td><td colspan="2">All vehicle types</td><td>15 mph</td></tr>
</table>

In Illinois, the speed limit for 90% of interstate miles was raised from 65 to 70 mph at the beginning of 2014. These roadways are made up of mostly rural highways, with the exception of five short sections of the Illinois Tollway, which were increased 15 to 70 mph. The Illinois DOT conducted traffic engineering studies and examined 85th percentile speeds in support of these recommendations.

Illinois actually takes 85th percentile speed into consideration when the speed limit is posted. After the general speed limits are selected from Table 2 they may be altered based on the engineering study and analysis of the current speed distribution of free-flowing vehicles. To alter the speed limits, the prevailing speed needs to be calculated. The prevailing speed is the computed average of 85th percentile speed, the upper limit of the 10 mph pace (which is the 10 mph range containing the most vehicles) and the average test run speed, all measured during free-flowing traffic conditions. Subject to further adjustments resulting from the anticipated violation rate, the prevailing speed, to the nearest 5 mph, can be used as the altered speed limit. The policy could control the operating speed by altering the speed limits. However, the policy does not consider the discrepancy between the operating speed and the design speed.

2.3.2. Other Countries

Forbes (2012) summarized several approaches to setting speed limits. According to his research, the speed limit in Canada and New Zealand was based on the function of the road and the adjacent land use and can be adjusted based on the traffic conditions and crash history. Sweden and Netherlands set the speed limits according to the crash types that are likely to occur, the impact forces that result and the tolerance of the human body to withstand these forces. Australia uses a computer program that uses knowledge and inference procedures that simulate the judgment and behavior of speed limit experts to select the speed limits.

It was noted that international crash rates were generally higher than those in the United States, primarily because proportionately more travel occurs on freeways in the United States, which were safer than other types of roads.

2.4 DESIGN CONSISTENCY EVALUATION

To quantify the design inconsistency, Lamm et al. (1999) proposed three design consistency evaluation criteria (Table 3) in which differences between design speed and operating speed, between operating speeds on successive roadway sections, and between assumed side friction and the demanded side friction were checked.

Table 3: Design Consistency Evaluation Criteria

Consistency Level	Criterion 1 (km/h)	Criterion 2 (km/h)	Criterion 3
Good	$\lvert V_{85} - V_d \rvert \leq 10$	$\lvert V_{85i} - V_{85\,i+1} \rvert \leq 10$	$f_R - f_{RD} \geq +0.01$
Fair	$10 < \lvert V_{85} - V_d \rvert \leq 20$	$10 < \lvert V_{85i} - V_{85\,i+1} \rvert \leq 20$	$+0.01 > f_R - f_{RD} \geq -0.04$
Poor	$\lvert V_{85} - V_d \rvert > 20$	$\lvert V_{85i} - V_{85\,i+1} \rvert > 20$	$-0.04 > f_R - f_{RD}$

where V_{85} is the operating speed, V_d is the design speed, V_{85i} and $V_{85i+1}$are the operating speeds on section *i* and *i*+1, $and f_R$ and f_{RD} are the side friction assumed and demand on section *i*.

Fitzpatrick et al. (2000) also proposed an evaluation criterion based on alignment indices. The indices include average radius (AR), ratio of maximum radius to minimum radius (RR), average rate of vertical curvature (AVC), and the ratio of radius of a single horizontal curve to the average radius of the entire section (CRR). Analysis of collisions on two-lane rural highways in Washington showed there was a significant connection between the collision frequency and these indices.

2.5 POTENTIAL SOLUTIONS

Based on the evaluation criteria above, the inconsistency could be identified. To find solutions to address the inconsistency in design speed and operating speed, various researchers have made efforts to include the operating speed in the design. The potential solutions could be the adoption of anticipated posted or operating speed plus a preset incremental increase as well as using a feedback loop with a speed prediction model (Fitzpatrick et al. 2003).

Some states currently tried to use the design speed with 5 or 10 mph addition to the posted speed limit to ensure the adequacy of design speed. It has been found that the decrease of speed limit could result in a less-than-equivalent change in average speed (Kockelman et al. 2006). However, Fitzpatrick et al. (2003) reported that the 5 or 10 mph addition to the speed limit could be insufficient in some rural areas. In addition to doubts about the effect of this procedure, its feasibility was a concern because it could result in different posted speed limits in areas with similar functional classes.

The feedback loop method has been adopted by several European countries and Australia, where the operating speed was incorporated in the design. Krammes et al. (1995) recommended the second method to be considered by AASHTO, especially in rural areas where design speeds are usually less driver speeds. The approach is summarized in four steps as follows:

1. Design a preliminary alignment based on a given design speed
2. Estimate the operating speeds (85th percentile speeds) on the alignment
3. Check the difference between operating speed on successive curves
4. Revise the geometric alignment to narrow the gap to acceptable levels

Based on the feedback loop, we conducted statistical and simulation analyses to examine the safety impact of design speed choices. Our methodology is presented in the flowchart in Section 3.

2.6 OPERATING SPEED PREDICTION MODELS

2.6.1 Operating Speed Summary

Given the design speed, it is possible to estimate the range of different geometric design elements using the equations from the AASHTO *Green Book*. The next step is predicting the operating speed based on these geometric features.

Several models have been established to predict the operating speed. A comprehensive summary of models for operating speed prediction is provided in Table 4. The accuracy and feasibility of these models could differ widely due to different driving behaviors and policies.

Table 4: Previously Developed Speed Prediction Models (Misaghi and Hassan 2005)

Author	Vehicle type	Model; Coefficient of determination
Taragin (1954)	PC	$V_{85}=88.87-2{,}554.76/R$; 0.860
McLean (1978)	PC	$V_{85}=101.2-0.0675\text{CCR}=101.2-2{,}730/R$; 0.870
McLean (1979)	PC	$V_{85}=53.80+0.464V_F-3{,}260/R+85{,}000/R^2$; 0.920
Kerman et al. (1982)	PC	$V_{85}=V_a-V_a^3/398R$; 0.910
Guidelines for the design of roads (1984)	PC	$V_{85}=60+39.70e^{-0.00358CCR}$ [LW=3.5 m]; 0.790
Glennon et al. (1986)	PC	$V_{85}=103.96-(4{,}524.94/R)$; 0.840
Setra (1986)	PC	$V_{85}=\{102/1+346/[(57{,}300/\text{CCR})^{-1.5}]\}$; N/A
		$V_{85}=88.72-0.084\text{CCR}$[LW=3.0 m]; 0.846
		$V_{85}=89.55-(2{,}862.69/R)$[LW=3.0 m]; 0.753
		$V_{85}=92.69-0.080\text{CCR}$[LW=3.3 m]; 0.731
		$V_{85}=93.83-(2{,}955.40/R)$[LW=3.3 m]; 0.746
Lamm and Choueiri (1987)	PC	$V_{85}=95.77-0.076\text{CCR}$[LW=3.6 m]; 0.836
		$V_{85}=96.15-(2{,}803.70/R)$[LW=3.6 m]; 0.824
		$V_{85}=94.39-(3{,}188.57/R)=93.85-0.045\text{CCR}$; 0.787
		$V_{85}=55.84-(2{,}809.32/R)+0.634\text{ LW}+0.053\text{ SW}+0.0004\text{ AADT}$; 0.842
Kanellaidis et al. (1990)	PC	$V_{85}=109.09-(3{,}837.55/R)$; 0.647
		$V_{85}=32.20+0.839V_d+(2{,}226.9/R)-(5{,}33.6/\sqrt{R})$; 0.925
		$V_{85}=129.88-(6{,}23.1/\sqrt{R})$; 0.777
Lamm (1993)	PC	$V_{85}=(10^6/8{,}270+7.20\text{CCR})$; 0.730
Ottesen and Krammes (1994)	PC	$V_{85}=103.04-0.0477\text{CCR}=103.70-(3{,}403/R)$; 0.800
Morrall and Talarico (1994)	PC	$V_{85}=e^{(4.561-0.00586DC)}$; 0.631
		$V_{85}=95.41-1.48\text{DC}-0.012(\text{DC})^2$[point of curve]; 0.990
Islam and Seneviratne (1994)	PC	$V_{85}=103.30-2.41\text{DC}-0.029(\text{DC})^2$[middle of curve]; 0.980
		$V_{85}=96.11-1.07\text{DC}$[point of tangency]; 0.980
		$V_{85}=103.66-1.95\text{DC}$; 0.800
Krammes et al. (1995)	PC	$V_{85}=102.45-1.57\text{DC}+0.0037L_C-0.10\text{DF}$; 0.820
		$V_{85}=41.62-1.29\text{DC}+0.0049L-0.12\text{DF}+0.95V_T$; 0.900
Lamm et al. (1995c)	PC	$V_{85}=(10^6/10{,}150.1+7.676\text{CCR})$; 0.810
Choueiri et al. (1995)	PC	$V_{85}=91.03-0.050\text{CCR}$; 0.810
		$\Delta V_{85PC}=3.64+1.78\text{DC}$; 0.510
		$\Delta V_{85LT}=2.00\text{DC}$; 0.690
		$\Delta V_{85T}=4.32+1.44\text{DC}$; 0.420
Al-Masaeid et al. (1995)	PC, LT, HT	$\Delta V_{85\text{ ALL}}=3.30+1.58\text{DC}$; 0.620
		$\Delta V_{85\text{ ALL}}=1.84+1.39\text{DC}+4.39\text{Pcon}+0.07G^2$; 0.770
		$\Delta V_{85\text{ ALL}}=(5{,}081/R_2)-(5{,}081/R_1)$[continuous curves]; 0.810
		$\Delta V_{85\text{ ALL}}=108.30-(3{,}498/L_T)-0.71[(\text{DF}_1\times\text{DF}_2)/(\text{DF}_1+\text{DF}_2)]$[common tangents]; 0.720
Voigt (1996)	PC	$V_{85}=99.61-(2{,}951.37/R)$; 0.840
Abdelwahab et al. (1998)	PC	$\Delta V_{85}=0.9433\text{DC}+0.0847\text{DF}$; 0.920
Pasetti and Fambro (1999)	PC	$V_{85}=103.90-(3{,}020.50/R)$; 0.680
Fitzpatrick et al. (2000)	PC	$V_{85}=106.30-(3{,}595.29/R)$[HC: $0\leq G<4$, or HC+sag VC]; 0.920
		$V_{85}=96.46-(2{,}744.49/R)$[HC: $4\leq G<9$]; 0.560
		$V_{85}=100.87-(2{,}720.78/R)$[HC: $-9\leq G<0$]; 0.590
		$V_{85}=101.90-(3{,}283.01/R)$[HC+LSD crest VC]; 0.780
		$V_{85}=111.07-(175.98/K)$[LSD crest VC]; 0.540
		$V_{85}=100.19-(126.07/K)$[HT+sag VC]; 0.680
Ottesen and Krammes (2000)	PC	$V_{85}=102.44-1.57\text{ DC}-0.012L_C-0.01\text{ DC}\times L_C$; 0.810
Andueza (2000)	PC	$V_{85}=98.25-(2{,}795/R)-(894/R_a)+7.486\text{ DC}+9.308L_T$[horizontal curve]; 0.840
		$V_{85}=100.69-(3{,}032/R)+27.819L_T$[tangent]; 0.850
McFadden and Elefteriadou (2000)	PC	$85\text{ MSR}=-14.90+0.144V_{85T}-(954.55/R)+0.0153L_T$; 0.712
		$85\text{ MSR}=-0.812+(998.19/R)+0.017L_T$; 0.603
Gibreel et al. (2001)	PC	$V_{85S1}=91.81+0.01\,0R+0.468\sqrt{L_V}-0.006G_1^3-0.878\ln(A)-0.826\ln(L_0)$[AT, sag]; 0.980

Author	Vehicle type	Model; Coefficient of determination
		$V_{85S2}=47.96+7.216\ln(R)+1.534\ln(L_V)-0.258\,G_1-0.653\,A+0.02\,e^E-0.008\,L_0$[BC,sag]; 0.980 $V_{85S3}=76.42+0.023\,R+0.00023\,K^2-0.008\,e^A+0.062\,e^E-0.00012\,L_0^2$[MC,sag]; 0.940 $V_{85S4}=82.78+0.011\,R+2.068\ln(K)-0.361\,G_2+0.036\,e^E-0.00011\,L_0^2$[EC,sag]; 0.950 $V_{S5}=109.45-1.257\,G_2-1.586\ln(L_0)$[DT,sag]; 0.790 $V_{85C1}=82.29+0.003\,R-0.05\,\Delta_C+3.441\ln(L_V)-0.533\,G_1+0.017\,e^E-0.000097\,L_0^2$[AT,crest]; 0.940 $V_{85C2}=33.69+0.002\,R+10.418\ln(L_V)-0.544\,G_1+[8.699/\ln(1+A)]+0.032\,e^E-0.011\,L_0$[BC,crest]; 0.970 $V_{85C3}=26.44+0.251\sqrt{R}+10.381\ln(L_V)-0.423\,G_1+[6.462/\ln(1+A)]+0.051\,e^E-0.028\,L_0$[MC,crest]; 0.980 $V_{85C4}=74.97+0.292\sqrt{R}+3.105\ln(K)-0.85\,G_2+0.026\,e^E-0.00017\,L_0^2$[EC,crest]; 0.900 $V_{85C5}=105.32-0.418\,G_2-0.123\sqrt{L_0}$[DT,crest]; 0.830
Jessen et al. (2001)	PC	$V_{85}=86.80+0.297V_P-0.614G_1-0.00239$ADT[LSD]; 0.540 $V_{85}=72.10+0.432\,V_P-0.00212$ADT[NLSD]; 0.420 $V_{85(1)}=56.1+0.117R-1.15G_1+0.006L_{T1}-0.000097(L_{T1})(R)$; 0.613
Donnell et al. (2001)	Trucks	$V_{85(2)}=78.4+0.0140R-1.40G_2-0.00724L_{T2}$; 0.562 $V_{85(3)}=75.1+0.0176R-1.48G_2-0.00836L_{T2}$; 0.600 $V_{85(4)}=74.5+0.0176R-1.69G_2-0.00810L_{T2}$; 0.611 $V_{85(5)}=83.1-2.08G_2-0.00934L_{T2}$; 0.577

Notation

The following symbols are used in this paper:

A = algebraic difference of vertical grades (%);
AADT;ADT = average annual daily traffic and average daily traffic, respectively (vehicles/day);
ANOVA = analysis of variance;
AT; DT = approach and departure tangent, respectively;
BC;EC;MC = beginning, end, and middle of curve, respectively;
CCR = curvature change rate (degree/km);
curve-dir = curve direction (right-turn: curve-dir=1; right-turn: curve-dir=0);
DC = degree of curvature (degrees);
DF = deflection angle (degrees);
$DF_1;DF_2$ = deflection angle for curves 1 and 2 of compound curve, respectively (degree);
DFC = deflection angle of circular curve (degrees);
drv-flag = driveway flag (intersection on curve: drv-flag =1; otherwise: drv-flag=0);
$e;E$ = superelevation rate (%);
G = vertical grade (%);
$G_1;G_2$ = first and second grade in direction of travel, respectively (%);
HT; LT = heavy truck and light truck, respectively;
int-flag = intersection flag (intersection on curve: int-flag=1; otherwise: int-flag=0);
K = length of vertical curve for 1% change in grade (m);
L_C = length of horizontal circular curve (m);
L_T = length of tangent (m);
$L_{T1};L_{T2}$ = length of preceding and succeeding tangent, respectively (m);
L_V = length of vertical curve (m);
L_0 = distance between horizontal and vertical points of intersection (m);
LW; SW = lane and shoulder width, respectively (m);
85 MSR = maximum speed reduction from tangent to middle of curve (km/h);
PC = passenger car;
Pcon = pavement condition (PSR$\geqslant$3: Pcon=0; otherwise: Pcon=0);
preradius; suc-radius = preceding and succeeding curve radius, respectively;
R = radius of the curve (m);
R_a = radius of previous curve (m);
sp-flag = spiral flag (curve with spiral: sp-flag=1; otherwise: sp-flag=0);
SPSS = statistical package for social sciences;
V_a = curve approach speed (km/h);
V_d = desired speed (km/h);
V_F = approach tangent speed (km/h);
V_P = posted speed limit (km/h);
V_T = approach tangent speed (km/h);
V_{85} = 85th percentile speed (km/h);
V_{85MC} = 85th percentile speed at middle of curve (km/h);
$\Delta_{85}V$ = 85th percentile speed differential calculated as 85th percentile value of speed differentials of individual drivers; and
ΔV_{85} = 85th percentile speed differential calculated as difference between V_{85} on two elements.

Contrary to the traditional 2D model, Gibreel et al. (2001) used a new approach and developed 3D models that combine the horizontal and vertical alignment. Two types of 3D models were established: (1) a combination of a horizontal curve and a sag vertical curve and (2) a horizontal curve combined with a crest vertical curve. By giving different weights to various roadway geometric factors, the model could be used to predict operating speed for five points in the curve (Figure 4). The result of the regression analysis showed a large gap between the 2D and 3D models, and the author recommended the 3D model because it fit the actual data better. The equations are as follows:

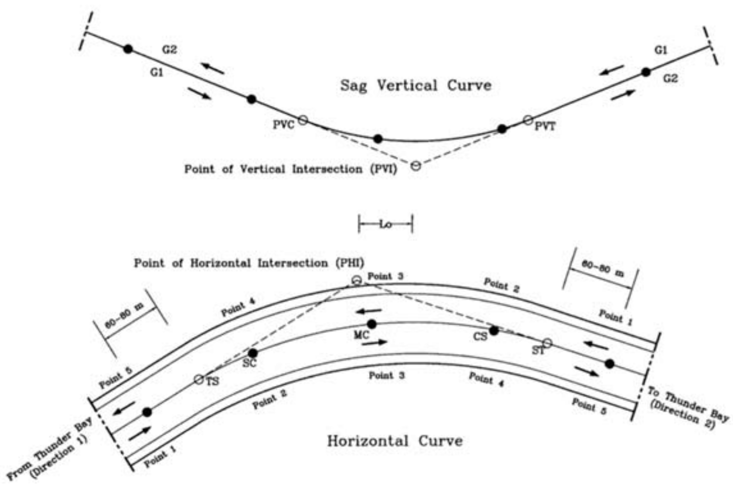


Figure 4: Distribution of observation points on a typical 3D combination (Gibreel et al. 2001).

$$V_S1 = 91.81 + 0.010r + 0.468\sqrt{L_V} - 0.006G_1^3 - 0.878\ln(A) - 0.826\ln(L_0) \quad (1a)$$
$$V_S2 = 47.96 + 7.217\ln(r) + 1.534\ln(L_V) - 0.258G_1 - 0.653\text{A} - 0.008L_0 + 0.020\exp(e) \quad (1b)$$
$$V_S3 = 76.42 + 0.023\text{r} + 2.300 \times 10^{-4}K^2 - 0.008\exp(A) - 1.230 \times 10^{-4}{L_0}^2 + 0.062\exp(e) \quad (1c)$$
$$V_S4 = 82.78 + 0.011\text{r} + 2.067\ln(K) - 0.361G_2 + 0.036\exp(e) - 1.091 \times 10^{-4}{L_0}^2 \quad (1d)$$
$$V_S5 = 109.45 - 1.257G_2 - 1.586\ln(L_0) \quad (1e)$$

where

$$V_S1 \ to \ V_S5 = \text{predicted 85th percentile operating speed at point 1 to point 5 (km/h)}$$

Point 1 was set at 60 to 80 m on the approach tangent before the beginning of the spiral curve. Point 2 was the end of spiral curve and the beginning of horizontal curve in the direction of travel. Point 3 was the middle point of horizontal curve. Point 4 was the end of the horizontal curve and the beginning of spiral curve in the direction of travel. Point 5 was set out at 60 to 80 m on the departure tangent after the end of the spiral curve.

r = radius of horizontal curve (m)

L_v = length of vertical curve (m)

e = super-elevation rate (%)

A = algebraic difference in grades (%)

K = rate of vertical curvature (m)

G_1 and G_2 = first and second grades in the direction of travel in percent (positive for upgrade and negative for downgrade)

L_0 = horizontal distance between point of vertical intersection and point of horizontal intersection (m)

R^2 = coefficient of determination

Similarly, the equations used for crest combination are as follows:

$$V_c1 = 82.29 + 0.003r - 0.05\Delta_c + 3.44\ln(L_V) - 0.553G_1 + 0.017\exp(e) - 0.00097{L_0}^2 \quad (1f)$$
$$V_c2 = 33.69 + 0.002r + 10.418\ln(L_V) - 0.544G_1 + 8.699 * \frac{1}{\ln(A+1)} + 0.032\exp(e) - 0.01L_0 \quad (1g)$$
$$V_c3 = 26.44 + 0.251r^{0.5} + 10.381\ln(L_V) - 0.423G_1 + 6.462 * \frac{1}{\ln(A+1)} + 0.051\exp(e) - 0.028L_0 \quad (1h)$$
$$V_c4 = 74.97 + 0.292r^{0.5} + 3.105\ln(K) - 0.85G_2 + 0.026\exp(e) - 0.00017{L_0}^2 \quad (1i)$$
$$V_c5 = 105.32 - 0.418G_2 - 0.123{L_0}^{0.5} \quad (1j)$$

where Δ_c= deflection angle of horizontal curve, and the other terms are the same as above.

Another report conducted in 1995 by Krammes et al. incorporated the speed on a tangent to estimate the operating speed on a curve. Results showed that the availability of the operating speed on a tangent could improve the performance of models for predicting operating speed.

$$V85 = 41.62 - 1.29D + 0.0049L - 0.12I + 0.95V_t \quad (2)$$

where

$V85$ = 85th percentile speed on curve (km/h)

V_t = 85th percentile speed on approach tangent (km/h)

D = degree of curvature (°)

L = length of curvature (m)

I = deflection angle (°)

Most of the existing operating speed models were developed using linear regression with a variety of geometric and traffic characteristics of the alignment (Hassan et al. 2011). The main independent variables included in the models were the radius of the curve and approach speed (the speed at which vehicle approaches a curve). However, a recent work by Vukoje et al. (2014) used a multiple regression procedure to establish a horizontal curve speed model. Based on the collected data, the following model was developed:

$$\hat{V}_{85} = 0.023R + 0.471V_{approach\,85} + 35.443 \tag{3}$$

where R is the curve radius. The coefficient of determination for the model is 0.761.

Some road design guidelines, like those in Germany and Australia, use the 85th percentile operating speed obtained by field survey. Other guidelines, such as those in Croatia, use the project speed (V_p) rather than the 85th percentile operating speed (Vukoje et al. 2014). Project speed is defined as the maximum expected speed in free-flow conditions that can be achieved with sufficient safety on a particular part of the road segment depending on its horizontal and vertical characteristics. It is determined from the following equation:

$$V_P = \sqrt{127 \cdot R \cdot (e_{max} + f_{Rdesign})} \tag{4}$$

where V_p is project speed (km/h), R is curve radius (m), e_{max} is maximum permissible super-elevation, and $f_{Rdesign}$ is design side-friction factor.

2.6.2 Operating Speed on Tangent

The operating speed on tangent sections is more difficult to predict due to the lack of available databases. Polus et al. (2000) collected numerous amounts of data from 162 sites. After analyzing the data, they used the geometric measure of the tangent section and attached curves for long tangent lengths as primary variables, then they calibrated the coefficient to build prediction models. To achieve a higher degree of reliability, they divided the alignment into four groups based on different geometric characteristics (e.g., tangent length, the radius of horizontal curvature). The models are as follows:

$$V_{85} = 101.11 - 3420/GM_S \qquad (R_{1,2} \leq 250\ m, TL < 150\ m) \tag{5a}$$
$$V_{85} = 98.405 - 3184/GM_S \qquad (R_{1,2} \leq 250\ m, 150\ m < TL < 1000\ m) \tag{5b}$$
$$V_{85} = 105.00 - 28.107/e^{0.00108 \times GM_L} \qquad (R_{1,2} \geq 250\ m, 150\ m < TL < 1000\ m) \tag{5c}$$
$$V_{85} = 105.00 - 22.953/e^{0.00012 \times GM_L} \qquad (1000\ m \leq TL, reasonable\ R_{1,2}) \tag{5d}$$

where

V_{85} = 85th percentile speed (km/h)

$\mathrm{GM_s}$ = geometric measure for short tangent lengths (m)

$\mathrm{GM_L}$ = geometric measure of tangent section and attached curves for long tangent lengths ($\mathrm{m^2}$)

TL = tangent length (m)

R_1 and R_2 = previous and following curve radii (m)

Ottesen and Krammes (2000) claimed that the speed reduction from approach tangents to horizontal curves will increase accident rates, so they used models to estimate operating speed on curves and approach tangents.

Speed on curves:

$$V85 = 103.66 - 1.95D \tag{6a}$$
$$V85 = 102.44 - 1.57D + 0.012L - 0.01DL \tag{6b}$$

Speed on approach tangents:

$$V85 = 41.62 - 1.29D + 0.0049L - 0.12DL + 0.95V_t \tag{6c}$$

where

$V85$ = 85th percentile speed (km/h)
D = degree of curvature (°)
L = length of curvature (m)
V_t = 85th percentile speed on approach tangent (km/h)

Then Ottesen and Krammes (2000) proposed a speed-profile model to estimate the 85th percentile operating speed reduction from approach tangents to horizontal curves.

$$TL_c = (2V_f^2 - V85_1^2 - V85_2^2)/25.92a \tag{6d}$$

where

TL_c = critical tangent lenth (m)
V_f = 85th percentile (desired) speed on long tangents (km/h)
$V85_n$ = 85th percentile speed on curve n (km/h)
a = acceleration/deceleration rate = 0.85 $\mathrm{m/s^2}$

Ottesen and Krammes (2000) thought the model could be used to check for speed consistency on rural highway horizontal alignments with design speeds less than 62.14mph (100 km/h) and considered it as the basis of the feedback loop.

With the emergence of various new databases, researchers were able to establish more reliable models. Zuriaga et al. (2010) built three models and calibrated them using global positioning system (GPS) data, in which the continuity of data was utilized to achieve more accurate results. As opposed to the previous model, these models were found to perform better when comparing with actual data.

Operating speed models on curves:

$$V_{85} = 97.4254 - 3310.94/R \quad (7a)$$
$$V_{85} = 1/(0.00948323 + 0.0000136809 \times CCR) \quad (7b)$$
$$V_{85} = 102.048 - 3990.26/R \quad (7c)$$

Operating speed models on tangents:

$$V_{85T} = V_{85C} + (1 - e^{-\lambda L}) \cdot (V_{des} - V_{85C}) \quad (7d)$$

Deceleration models:

$$d_{85} = 0.263571 + 67.7999/R \quad (7e)$$
$$d_{85} = 0.242186 + 0.00150693/R \quad (7f)$$

where

$\mathrm{CCR} =$ curvature change rate (°/km)
$\mathrm{V_{des}} =$ desired speed (km/h), assumed 68.35mph(110 km/h) from the literature
$\mathrm{V_{85C}} =$ 85th percentile speed on previous curves obtained from the proposed model
$\mathrm{L} =$ tangent length (m)
$\mathrm{R} =$ curve radius (m)
$\lambda = 0.00135 + (R - 100) \cdot 7.00625 \cdot 10^{-6}$

2.7 CRASH RATE ESTIMATION

2.7.1 The Role of Speed in Crashes

Excessive speed was reported to be an important contributory factor in many crashes. It was also reported that approximately 62% of the nation's speeding-related fatalities were on rural roads.

Speed can contribute to a crash occurring if it is higher than the local speed limit or than the circumstances at that moment allow (e.g., rain). However, it is difficult to determine the number of crashes in which speed was the main cause because various other factors are often involved. Inappropriate speed is especially difficult to determine objectively. It is generally assumed that about one-third of fatal crashes are (partly) caused by excessive or inappropriate speed.

Regarding crash severity, collision speed is important to crash outcome. The higher the collision speed, the more serious the consequences in terms of injury and property damage. At a collision speed of 49.70mph (80km/h), the possibility that the car's occupants will be killed is about 20 times greater than at a speed of 18.64mph (30 km/h).

The road's design speed also has an influence on crashes. On a road with a design speed of 80 km/h, a speed increase from 49.70mph (80km/h) to 55.92mph (90 km/h) results in a larger increase in crash rates than the same increase on a road with a design speed of 62.13mph (100 km/h). This is a consequence of 49.70mph (80km/h) roads not being designed for these faster speeds (SWOV 2009).

Studies have generally shown that on rural, non-limited-access highways, no significant relationship exists between average traffic speeds and crash or fatality rates, while speed dispersion contributes significantly to increased crash probability for this road class.

The performance of highway safety can be quantitatively evaluated using such techniques as those provided by the AASHTO *Highway Safety Manual (HSM)* (AASHTO 2010) and the FHWA Crash Prediction Module (CPM) of the Highway Safety Design Model (HSDM). These models can be used to estimate or predict the expected number of road crashes on road segments and intersections. The general approach is to identify a relationship between road accidents and the contributing factors using various statistical models. Such a relationship was identified for either the individual road or area level. A number of recent studies suggested that area-based models were more appropriate than individual road-based models because the former takes into account system-wide effects (Barker et al. 1999; Noland and Oh 2004; Haynes et al. 2007).

Several roadway characteristics and other factors have been found to impact safety along two-lane, non-freeway segments (Gates et al. 2016):

- design speed and posted speed limit
- traffic and truck volumes
- horizontal and vertical alignment
- lane width and surface type
- shoulder type and width

There are two principal features in the crash prediction algorithm: safety performance functions (SPFs) and crash modification factors (CMFs). The SPFs are used to estimate the predicted number of total crashes for base conditions based on the crash history of roadways with similar attributes (e.g., two-lane rural highway), and CMFs are used to adjust these base predictions to reflect the features of a specific roadway. The SPFs estimations are constructed based on count regression models. In the base condition, the SPFs for a two-lane rural highway consider the roadway to be a horizontal tangent with a level grade and 12 ft. (3.6 m) travel lanes. A sample SPF for a two-lane rural highway roadway segment is as follows (Harwood et al. 2000):

$$NBR = ADT \times L \times 10^{-6} \times e^{-0.4865} \tag{8}$$

where

NBR = predicted number of total highway element crashes per year

ADT = average daily traffic volume (vehicle/day)

L = length of highway element (km)

Garber and Ehrhart (2000) categorized the crash-affecting factors as speed characteristics, traffic characteristics, and geometric characteristics.

2.7.2 Speed Characteristics

Previous research showed that absolute speed and speed dispersion were two factors highly correlated to crash rate (Aarts and Van Schagen 2006).

2.7.2.1 Absolute Speed

In rural areas, roadway design speed is often based on the most restrictive geometric element, usually a horizontal or vertical curve. Vehicular operating speeds along tangent sections of two-lane highways have been shown to be impacted by the posted speed limit, with vehicular speeds tending to increase as the posted speed limit increases (Kockelman et al. 2006)

Relatively many studies have examined the relation between absolute speed and the crash rate. Irrespective of the research method used, practically all the studies concluded that the relation between speed and crash rate can best be described as a power function (an exponential relation between speed and the crash rate): the crash rate increases *faster* when the speed increases and vice versa.

An increase or decrease in speed has a greater effect on serious crashes than on light crashes. Based on kinetic laws, Nilsson et al. (1982) calculated this in the early 1980s. The effect on the number of injury crashes can be expressed by the following formula, referred to as the power model:

$$LO_2 = LO_1 \left(\frac{v_2}{v_1}\right)^2 \quad (9)$$

with LO_2 being the number of injury crashes after the change in speed, LO_1 being the initial number of injury crashes, v_1 being the average speed before the change, and v_2 being the average speed afterward.

The same formula could be used to describe the effect on the number of crashes with severe injury—not to the 2nd power but to the 3rd; for fatal crashes, its effect was to the 4th. The power functions have been validated using more recent data (Nilsson 2004; Christensen et al. 2005). The researchers also applied the models to different road types. Using these formulas, the effect of speed changes was calculated for different speed limits and for different crash severities (Table 5). These percentages give an indication of the expected effects of a change in average speed of 1 km/h for different initial speeds. The real effect on a particular road can, of course, deviate from the outcome (for example, due to specific road or traffic features).

Table 5. Expected Effect of a Speed Change of 1 km/h on the Number of Crashes of Different Severities at Different Initial Speeds (Source: Aarts and Van Schagen 2006)

	Reference Speed					
Crash Severity	**31.07 mph (50 km/h)**	**43.50 mph (70 km/h)**	**49.70 mph (80 km/h)**	**55.92 mph (90 km/h)**	**62.14 mph (100 km/h)**	**74.56 mph (12 0km/h)**
Injury crashes (%)	4.07	2.9	2.5	2.2	2.0	1.7
Severe injury (%)	6.1	4.3	3.8	3.4	3.0	2.5
Fatal crashes (%)	8.2	5.9	5.1	4.5	4.1	3.3

Generally, accidents were more serious at higher speeds. *S*everal researchers have investigated the relationship between speed and road accidents (Shefer and Rietveld 1997; Aljanahi et al. 1999; Ossiander and Cummings 2002; Kockelman and Ma 2007). These studies were often based on a disaggregated individual road-level speed. In some cases, the relationship between mean speed and accident rates was significant. Ossiander and Cummings (2002) examined the change of the freeway speed limit in Washington State using time series data and found that an increased speed limit, which could result in higher operating speed, was associated with a higher fatality rate.

Self-report studies and case-control studies were widely used to determine the relationship between speed and crash rates. Fildes et al. (1991) adopted a self-report study, in which they utilized interviews with drivers and the speed measurement devices to analyze different factors that influence crash rates. In their report, the researchers concluded that crash rates tend to increase with higher absolute speed. However, there are certain disadvantages in self-report studies due to the lack of information about speed before the crash.

Kloeden et al. (2001) used the case-control method to quantify the relationship between speed and crashes. They selected several case vehicles under certain criteria and investigated the information of the crashes. Afterward, the Simulation Model of Automobile Collisions (SMAC) was used to reconstruct the crash scene and the crash risk curve was calculated through modified logistic regression model. Later research by Kloeden et al. (2002) established the mathematical relationship to illustrate the crash risk curve. Based on that model, the crash risk tends to increase with the increase of absolute speeds.

$$relative\ risk(V) = e^{-0.822957835-0.083680149V+0.001623269V^2} \tag{10}$$

where V = free-traveling speed in km/h, a free-traveling speed is defined as the speed of a vehicle moving along a mid-block section of road, or with right-of-way through an intersection, and not slowing to join, or accelerating away from a traffic stream.

As an example of how this equation is applied, a vehicle that travels in a 37.28 mph speed zone at a speed of 43.5 mph will have a risk of being involved in a casualty crash that is 3.6 times greater than a vehicle that travels at 37.28 mph. Note that this estimate of the relative risk only applies to vehicles that are traveling at a free speed. Risk estimates may be derived from the above equation for a range of speed.

A recent study by Kockelman and Ma (2007) examined the freeway speed and speed variation preceding crashes in California while controlling for other factors such as weather and lighting conditions. They found no evidence that speed condition influences crash occurrence. This may be due to data aggregation and crash-time reporting errors, as suggested by the authors. Their study was also based on disaggregated road-level data.

2.7.2.2 Speed Deviation

In addition to absolute speeds, the speed differences between vehicles also have an effect on crash rates. Aljanahi et al. (1999) and Ossiander and Cummings (2002) suggested that it is also necessary to investigate speed variance to see how it relates to road casualties because it also plays an important role.

The first type of studies that concentrated on speed differences between individual vehicles involved in a crash and all other vehicles were conducted in the United States in the 1950s and 1960s (e.g., Solomon 1964). These studies always found a U-curve: the slower or faster a car drives compared with most of the vehicles on that road, the more the risk of being involved in a crash increased.

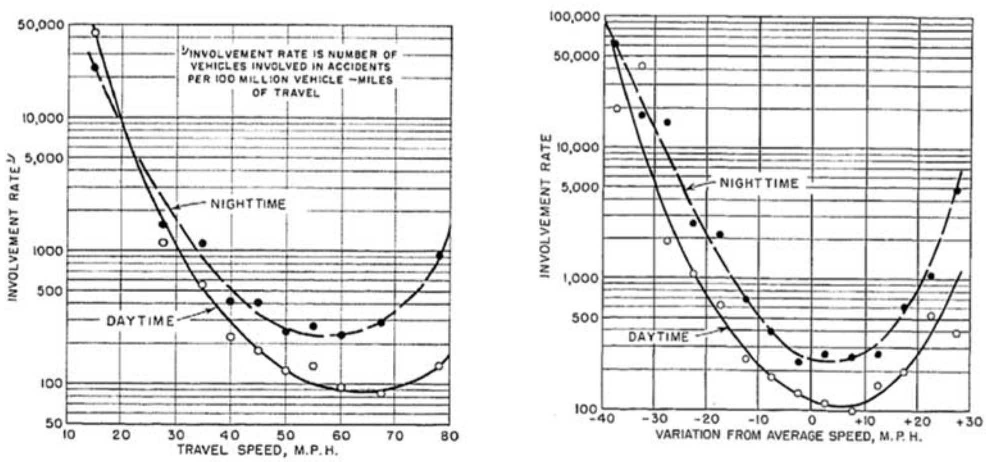


Figure 5: Involvement rate by variation from average speed on study section, day and night (Solomon et al. 1964).

Solomon et al. (1964) analyzed the accident records of nearly 10,000 drivers and the interviews of 29,000 drivers. Several methods for measuring speed and collecting data were also provided in that report. That research compared the speed distribution of accident-related drivers with drivers who were not involved in an accident. To relate the hazard to the travel speed of those two groups of drivers, Solomon et al. (1964) used the involvement rate, which was the number of involvements per 100 million miles of travel, as an indicator. On the basis of their observations of the involvement, the researchers suggested the famous U-shaped relationship for characterizing the relationship between involvement in crashes and the dispersion of speed. According to those results, the involvement rate, as an indicator of crash risk, increased when the speed deviates from the average speed, as shown in Figure 5. Also, the relationship between travel speed and involvement rate was illustrated in that study, as shown in Figure 6.

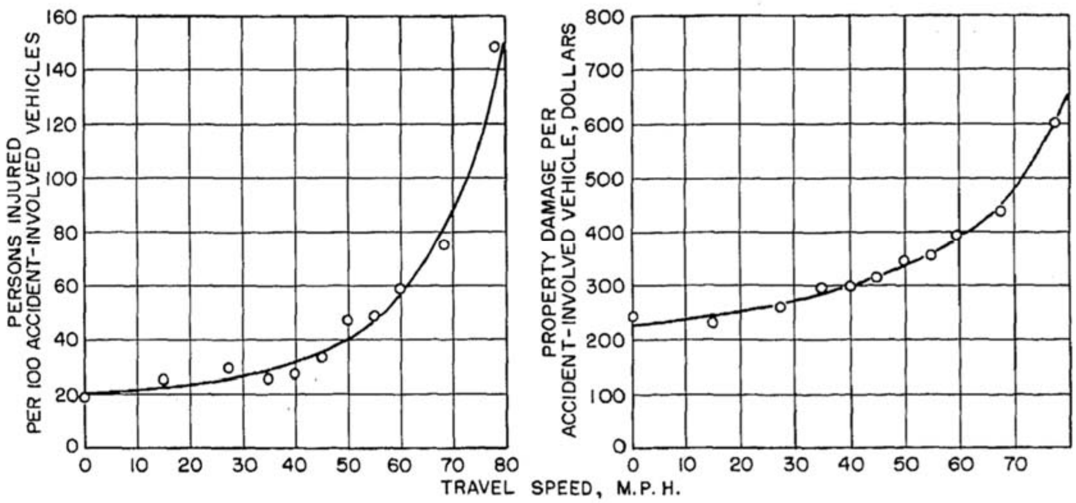


Figure 6: Persons injured per 100 involvements and property damage per involvement by travel speed during daytime hours (Solomon et al. 1964).

Cirillo (1968) also conducted similar research and their results were similar to those of Solomon et al. (1964). However, despite its significance, these studies have certain drawbacks such as inaccuracy of speed measurement and mismatches between the case and control vehicles (Shinar 1998.)

However, recent studies that used more modern measuring instruments and a more accurate research design reached a different conclusion. Kloeden et al. (2002) developed a model to relate crash risk to speed deviation through a case-control study and found that vehicles that were driven considerably faster than average on the road in question had a higher crash rate than vehicles that were driven slower. They established the following model:

$$relative\ risk(D) = e^{0.1133374D+0.002817D^2}$$

where D = difference in traveling speed in km/h.

As an example of how this equation is applied, a vehicle traveling in a 37.28mph speed zone at a speed 6.21mph faster than the average speed of the rest of the traffic at a particular location will have a risk of crashing that is 4.1 times greater than a vehicle traveling at the average speed of the rest of the traffic at that particular location (applies only to vehicles traveling at a free speed).

The relative risk estimates may be derived from the Equation 11 for speed differences. The model showed a pattern different from Solomon's U-shaped curve, but the model developed by Kloeden et al. (2002) still showed that the crash risk increases with higher speed difference.

Quddus (2013) explored the relationships among mean speeds, speed variations, and crash rate. The results suggested that mean speeds were not associated with crash rate when controlling for other factors affecting crashes, such as traffic volume, road geometry, and number of lanes. However, speed variation was found to be statistically and positively associated with the crash rate.

Using regression analysis, De Oña et al. (2013) developed a relationship to predict crashes between speed reduction and crashes for horizontal curves on two-lane rural roads in Spain. Exposure, curve length (CL), and difference in 85th percentile speeds (ΔV_{85}) between successive tangents and horizontal curves, as well as between successive curves, were used. They first used the following relationships to estimate the operating speed on curves:

$$V_{85} = 120.16 - 5{,}596.72/R \tag{12a}$$

$$V_{85} = 97.4254 - 3{,}310.94/R \text{ for } 400 \text{ m} < R \leq 950 \text{ m} \tag{12b}$$

$$V_{85} = 102.048 - 3{,}990.26/R \text{ for } 70 \text{ m} < R \leq 400 \text{ m} \tag{12c}$$

where *R* is the horizontal curve radius (m)

Next, they developed the following model to predict crashes:

$$Y = \exp(\beta_0)\, AADT^{\beta_1} CL^{\beta_2} \exp(\beta_3 \Delta V_{85}) \tag{13}$$

where *Y* is the estimated number of crashes in 3 years, *AADT* is the average annual daily traffic (vehicles/day), *CL* is the curve length (km), and $\beta_0, \beta_1, \beta_2$ are parameters.

2.7.2.3 Road Type

Both the size of the crash rate and the extent by which the crash rate increases at faster speeds strongly depend on the road type. Roughly speaking, motorways have the lowest crash rate and, as the speed increases, the crash rate for motorways increases less rapidly than that for lower-order roads. The reverse is also true: the same drop in speed (in km/h) has a larger safety effect on lower-order roads than on higher-order roads.

Rural roads and, to an even larger extent, urban roads, have much more complex traffic environments in comparison with motorways: there are encounters with more types of road users coming from different directions and therefore showing less predictable behavior. In addition, and partly related to it, the road's design speed also has an influence. On a road with a design speed of 49.71mph (80 km/h), a speed increase from 49.71 to 55.92 mph (80 to 90 km/h) results in a larger increase in crash rate than the same increase on a road with a design speed of 62.14mph (100 km/h). This is a consequence of 49.71mph (80 km/h) roads not being designed for these faster speeds.

To make an estimation of the effects of speeding measures on crash rates, the following must be taken into account:

- *absolute speed*: the relation between (absolute) speed and crash rate is not a linear but is a power function (average speed for a road section) or an exponential relation (individual vehicle speed)
- *speed differences*: greater speed differences result in a higher crash rate; if a measure results in a lower average speed, but simultaneously in greater speed differences, then the

ultimate safety effect can be smaller or even the opposite of the effect of only the average speed reduction

- *road type*: the absolute crash rate and its increase at faster speeds is higher in complex traffic situations than in less complex situations

2.7.3 Traffic Characteristics

Many researchers have tried to describe the relationship between traffic flow and crash rates using statistical methods, and they found it to be a non-linear function. Ceder (1982) and Martin (2002) observed the U-shaped relationship (Figure 7) between the crashes rate and the hourly traffic flow based on the data collected. The results showed higher crash rate when the hourly traffic flow was very low or relatively high. Brodsky and Hakkert (1983) also found that travel densities have a moderate positive effect on accident rate. They also observed high fatal accident rates in states with lower travel densities.

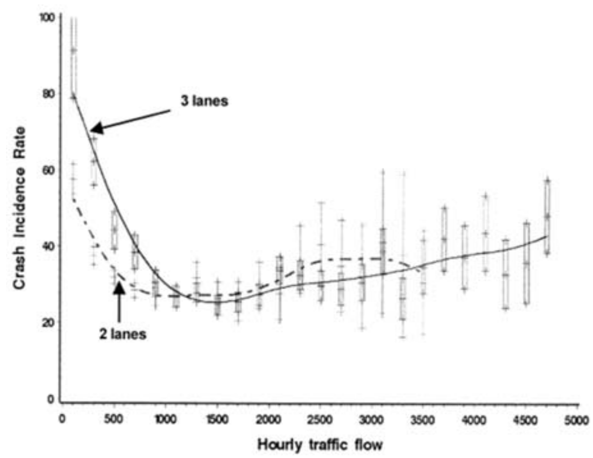


Figure 7: Crash incidence rates (crashes/100 million vehicle - km) per hourly traffic flow (Martin 2002).

2.7.4 Geometric Characteristics

2.7.4.1 Horizontal Curvature

Several studies have generally demonstrated that horizontal alignment is a primary factor in the vehicular operating speeds along two-lane highways because drivers tend to reduce speed based on the degree of curvature. Kockelman et al. (2006) suggested that geometric alignment has a greater influence over vehicular operating speeds than posted speed limits. They found that highway segments with horizontal curvature possessed higher crash rates than tangent segments. Similar findings were reported in a recent study that found that the presence of horizontal curvature tended to increase the rate of injury and fatal crashes but had no impact on property-damage crashes (Gates et al. 2015).

The work of Fitzpatrick et al. (2000) was more specific. They found that operating speeds on horizontal curves with a radius greater than or equal to 2,600 ft were similar to those on long tangents, although for radii below 800 ft, a sharp decrease in vehicle operating speed was observed.

FHWA's Interactive Highway Safety Design Model (IHSDM) includes a series of models to help predict the speed reduction likely to occur when traveling from a tangent segment to a horizontal curve (FHA). The *Highway Safety Manual* (2010) provides the following relationship for computing the crash modification factor for horizontal curves on rural two-lane highways:

$$CMF = [(1.55 \times L_C) + (80.2/R) - (0.012 \times S)]/(1.55 \times L_C) \quad (14)$$

where

L_C = length of horizontal curve including length of spiral transitions, if present (mile),
R = radius of curvature (ft),
S = 1 if spiral transition curve is present; 0 if spiral transition curve is not present.

The CMF applies to total crashes and the base condition consists of a tangent segment with no curvature.

Research that evaluated the impact of design parameters (degree and length of curve, super-elevation rate, lane width, sight distance, etc.) demonstrated that the most successful parameter in explaining the variability in accident rates (ACCR) was the degree of curve as quantified by the following regression models (Lamm et al. 1995):

12 ft lane width: $ACCR = -0.55 + 1.08 \cdot DC; R^2 = 0.73$ (15a)
10 ft lane width: $ACCR = -1.02 + 1.51 \cdot DC; R^2 = 0.30$ (15b)

where

$ACCR =$ estimate of accident rate including all accidents (acc./10^6 vehicle-miles).
$DC =$ degree of curve (degree/100 ft), range: 0° to 25°
$R^2 =$ coefficient of determination

Fink and Krammes (1995) showed that accident rates increase with a larger mean degree of curvature (Equation 16). Their analysis also illustrated that extreme approach conditions such as long and short approach sight distance can increase accident rates.

Mean accident rate = 0.05 + 0.23 * mean degree of curvature (16)

where mean accident rate is the number of accidents per 10^6 vehicle kilometers of travel.

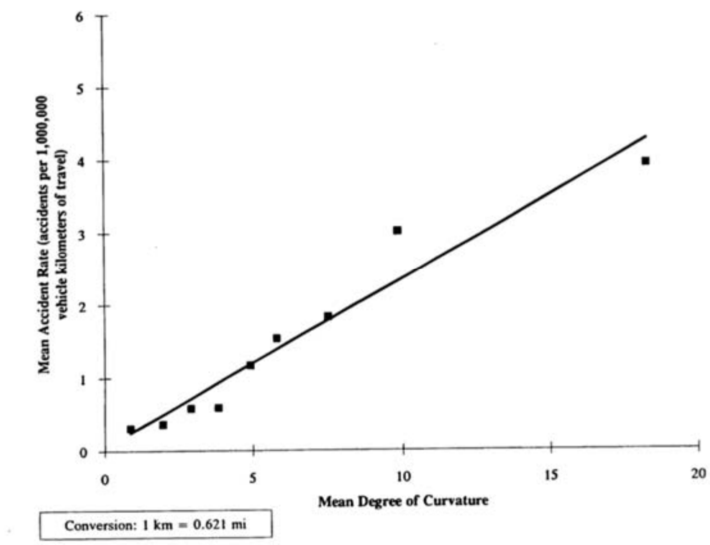


Figure 8: Mean accident rate versus mean degree of curvature (Fink and Krammes 1995).

An analysis of road curvature (i.e., the quantity of curved roads in an area) and road casualties was studied by Milton and Mannering (1998) and Haynes et al. (2007). Milton and Mannering (1998) developed a relationship pertaining to sections of highways and suggested that curved sections may not necessarily cause more traffic accidents. Haynes et al. (2007) studied road curvature and its association with traffic crashes at the district level (a census area) in England and Wales. Their studies developed a number of measures for road curvature and found that at the district level, road curvature was statistically significant with a negative coefficient—meaning that more curved roads in an area resulted in fewer road accidents. This may be explained by the fact that drivers would be more aware of the road conditions on a curved road. Evidence of this is that drivers usually operate a vehicle more carefully through residential and commercial areas, where the roads are often more curved. A recent study (Haynes et al. 2008) using similar methods applied in New Zealand showed that road curvature has an inverse relationship with fatal crashes in urban settings. However, this is not always true in rural areas.

2.7.4.2 Vertical Curvature

The safety effects of vertical curves are most likely related to limited stopping sight distance and type of terrain. Prior research (Kockelman et al. 2006; HSM 2010) found that segments with vertical curvature exhibited higher crash rates than relatively flat segments, and that research has also demonstrated that steeper vertical grades are associated with higher crash rates, although crashes on steeper vertical upgrades tended to be less severe.

2.7.4.3 Lane Width and Shoulder Width

Lane width and shoulder width are also two geometric characteristics that have an influence on the roadway safety (Garber and Ehrhart 2000). A study conducted by Garber et al. (1993) showed that trucks on the roads with smaller lane width have higher accident rates than those on the roads with larger lane width. However, some researchers did not observe such a relationship or found it insignificant (Dart and Mann 1970). Also, the relationship between shoulder width and crash rates is yet to be determined. Raff (1953) and Perkins (1957) claimed that the larger shoulder width could reduce crash rates, while Cirillo et al. (1969) did not observe that relationship. Noland et al. (2004) used county-level time series data from Illinois to analyze the relationship between changes in geometric design and road fatalities and reported accidents. They found that an increase in outside shoulder width could decrease the number of accidents. Also, they found that increasing lane width appears to be associated with increased fatalities.

The impact of lane width on traffic crashes varies with the associated traffic volume, and the effect is most pronounced for roadways involving lane widths of 9 ft or less. The effect of lane width on safety performance was noted to be lower for multi-lane highways than for two-lane highways.

Several studies have established the relationship between safety and shoulder width, and developed the CMF for shoulder width. Early in the 1980s, Zegeer et al. found that widening the shoulders from 1.6 ft to 8.2 ft (0.5 m to 2.5 m) on both sides of the road for an average section of a rural two-lane highway could reduce run-off-road and opposite-direction accidents by 16%. However, widening the shoulders to over 9 ft (2.7 m) is not economical.

Jovanis et al. (2007) conducted studies using case-control and cohort methods and estimated the odds ratio (which can be considered a CMF) and relative risk. Results from both methods showed that crashes decrease as shoulder width increases. The result from the case-control study showed that crash risks for shoulders narrower than 4 ft were very high, while there was no significant difference in crash rates for shoulder widths from 5 ft to 7 ft. The crash risk was very low when the shoulder width is greater than 7 ft. Similarly, the results from cohort methods showed almost the same trend.

To identify the safety effect of increased shoulder width or lane width given a fixed total pavement width, Gross et al. (2009) adopted the case-control method using the data from Pennsylvania and Washington and found a small benefit to increasing the lane width given a fixed total width. They found the CMFs obtained from their research were smaller than the CMFs in the *Highway Safety Manual*, showing that the joint effect of shoulder width and lane width should be considered in practice.

The frequency of traffic crashes tends to increase as paved shoulder widths are reduced below 6 ft. Increased shoulder widths have also been shown to increase operating speeds, likely due to the increased shy distance (Gates et al. 2015)

The Federal Highway Administration recommend the use of CMFs to measure the safety effectiveness of a particular treatment or design element. The crashes with treatment could be obtained by multiplying the crashes without treatment by the CMF.

The *Highway Safety Manual* (2010) provides the following relationship for computing the CMF for shoulder width and lane width:

$$CMF_{2r} = (CMF_{wra} \times CMF_{tra} - 1.0) \times p_{ra} + 1.0 \quad (17)$$

where

CMF_{2r} = crash modification factor for the effect of shoulder width and type on total crashes

CMF_{wra} = crash modification factor for related crashes (i.e., single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes), based on shoulder width

CMF_{tra} = crash modification factor for related crashes based on shoulder type

p_{ra} = proportion of total crashes constituted by related crashes (Illinois default value = 0.372)

The CMFs for shoulder width included in the *Highway Safety Manual* can be interpolated between the values from Table 6 and Table 7.

Table 6: Crash Modification Factors for Shoulder Widths on Roadway Segments (CMF_{wra}) (AASHTO 2010)

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

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

Table 7: Crash Modification Factors for Shoulder Types and Shoulder Widths on Roadway Segments (CMF_{tra}) (AASHTO 2010)

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

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

The CMFs for lane width included in *Highway Safety Manual* can be interpolated between the values from Table 8.

Table 8: Crash Modification Factors for Lane Width on Roadway Segments (CMF_{ra}) (AASHTO 2010)

	AADT (vehicles per day)		
Lane Width	**< 400**	**400 to 2000**	**> 2000**
9 ft or less	1.05	$1.05 + 2.81 \times 10^{-4}(AADT - 400)$	1.50
10 ft	1.02	$1.02 + 1.75 \times 10^{-4}(AADT - 400)$	1.30
11 ft	1.01	$1.01 + 2.5 \times 10^{-5}(AADT - 400)$	1.05
12 ft or more	1.00	1.00	1.00

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

2.7.5 Multivariate Models for Crash Rate Estimation

The univariate model may neglect the joint effect of different influential factors. The multivariate models could include the combination of various affecting factors and thus could provide a more accurate estimation under complicated conditions.

Garber and Ehrhart (2000) adopted the multiple linear regression method as well as the multivariate ratio of polynomial procedure to test combinations of different variables. These models were selected using two different criteria: Akaike's information criterion (AIC) and the coefficient of determination (R^2).

Four models that satisfy the two criteria were built. These models were very complicated with different powers of each entry. It is difficult to explain their physical meaning. Also, the data used to calibrate these models was limited to Virginia roads, but Garber and Ehrhart (2000) showed that they could still provide a relatively reliable evaluation of the road safety condition. Following are four models with different AIC and R^2.

Model 1:
$$\ln(\text{crash rate}) = 44.323 - 25755.82/SD^2 + 93793.11/SD^4 - 8.686*10^{-3}*FPL^2 + 0.106/SD^2*FPL^2 - 1.68710^{-8}*FPL^4 + 469.071/LW^{0.5} + 44529.25/SD^2/LW^{0.5} + (1.445*10^{-2}*FPL^2/LW^{0.5} - 956.114/LW^{0.5})^2 - 93.415*SW - 660.808/SD^2*SW + 5.626*10^{-5}*FPL^2*SW + 152.084/LW^{0.5}*SW + 3.475*SW^2,\ R^2=0.9864,\ AIC=-48.715 \tag{18a}$$

Model 2:
$$1/\text{crash rate}^{0.5} = 1132.667 - 3035.839/SD - 13380.54/SD^2 - 1.436*10^{-3}*FPL^2 - 4.313*10^{-2}/SD*FPL^2 + 1.752*10^{-7}*FPL^4 - 9519309/MEAN^2 - 6956803/SD/MEAN^2 - 71.254*FPL^2/MEAN^2 - 1.060174*10^9/MEAN^4 - 210.998*LW + 1963.584/SD*LW + 3.751*10^{-3}*FPL^2*LW + 3334646/MEAN^2*LW - 65.918*LW^2,\ R^2=0.9817,\ AIC=18.247 \tag{18b}$$

Model 3:
$$1/\text{crash rate}^{0.5} = 23635.61 - 17107.41/SD - 12605.73/SD^2 - 1.184*FPL - 10.318/SD*FPL + 2.621*10^{-3}*FPL^2 - 25345.75*LW^{0.5} + 10829.1/SD*LW^{0.5} + 1.051*FPL*LW^{0.5} + 6744.683*LW - 156.286*SW^2 + 199.0262/SD*SW^2 - 8.073*10^{-2}*FPL*SW^2 + 89.694*LW^{0.5}*SW^2 - 2.945*SW^4,\ R^2=0.9697,\ AIC=28.845 \tag{18c}$$

Model 4:
$$1/\text{crash rate} = 1331486 - 1365.183*SD - 5.771*SD^2 - 0.541*FPL^2 + 6.709*10^{-3}*SD*FPL^2 + 3.204*10^{-6}*FPL^4 - 4744279/LW^{0.5} + 2739.721*SD/LW^{0.5} + 0.873*FPL^2/LW^{0.5} + 4219166/LW + 1871.932*SW^2 - 23.51972*SD*SW^2 - 0.031*FPL^2*SW^2 - 2230.496/LW^{0.5}*SW^2 - 47.560*SW^4,\ R^2=0.9312,\ AIC=171.06 \tag{18d}$$

where

SD = standard deviation of speed (km/h)

FPL = flow per lane (vph)

LW = lane width (m)

SW = shoulder width (m)

MEAN = mean speed (km/h)

Ma et al. (2008) developed a multivariate Poisson-lognormal regression model for prediction of crash counts by severity, using Bayesian methods. They calibrated parameters in the following model.

$$y_{is}|\vec{\varepsilon}_i , \beta_s , x_{is} \sim Poisson(\lambda_{is}) \tag{19}$$

where

$\lambda_{is} = \exp(x_{is}{}'\beta_s + \varepsilon_{is})$

y_{is} = crash counts for roadway segment *i*, severity level *s*

$x_{is}{}'$ = severtiy level-specific explanatory variables

ε_{is} = severity-level-specific unobserved heterogeneity across roadway segments

For different levels of accident severity, the changes in crash rates with respect to road geometric elements (vertical/horizontal curve length, degree of curvature, vertical grade, shoulder width, etc.), traffic condition (AADT), and the functions of roads (arterial, collector etc.) are evaluated (Table 9).

Table 9: Expected Percentage Changes in Crash Rates Corresponding to Changes in Variables (Ma et al. 2008)

Variables	Averages	Changes in variable	Percentage change in crash rates (per 100 million VMT)					
			Fatal (%)	Disabling (%)	Non-disabling (%)	Possible (%)	PDO (%)	Total (%)
CURV_LGT	248 (ft)	+100	−0.36	0.65	−0.20	–	–	0.30
DEG_CURV	2.3 (°/100 ft)	+2	4.08	3.98	27.04	18.63	21.98	18.58
VCUR_LGT	303 (ft)	+100	0.37	−3.76	−2.06	−3.01	−2.08	−2.52
PCT_GRAD	1.805	+2	−12.41	24.88	30.93	27.62	24.07	24.86
SHLDWID	2.1 (ft)	+5	–	–	−5.54	−6.49	−7.89	−7.04
SURF_WID	24 (ft)	+5	−12.52	−58.65	−5.36	−6.49	4.76	0.04
SPD_LIMT	50 (mile/h)	+10	28.97	38.56	−12.72	25.64	−1.95	12.99
AADT	3757	+1000	–	41.37	–	10.24	4.68	16.42

where

CURV_LGT = horizontal curve length (ft)

DEG_CURV = degree of curvature (°/100 ft)

VCUR_LGT = vertical curve length (ft)

PCT_GRAD = vertical grade

SHLDWID = shoulder width (ft)

SURF_WID = surface width (ft)

SPD_LIMT = speed limit (mph)

AADT = average annual daily traffic

2.7.6 Crash Rate Estimation Based on Speed Profile

The tangent-to-curve transition was one of the most critical locations when safety condition was evaluated (Camacho-Torregrosa et al. 2013). According to Lamm et al. (1992), more than 50% of fatalities on rural roads take place at curve sections. Anderson et al. (1999) tested the relationship between accidents and speed reduction, based on data collected between 1993 and 1995 from the Washington Highway Safety Information System (HSIS). Qualitative and quantitative analyses were conducted. Speed reduction was found to be highly correlated with accidents and could be considered a very promising consistency measure. Castro et al. (2008) illustrated the speed profile on the tangent-to-curve transition (Figure 9).

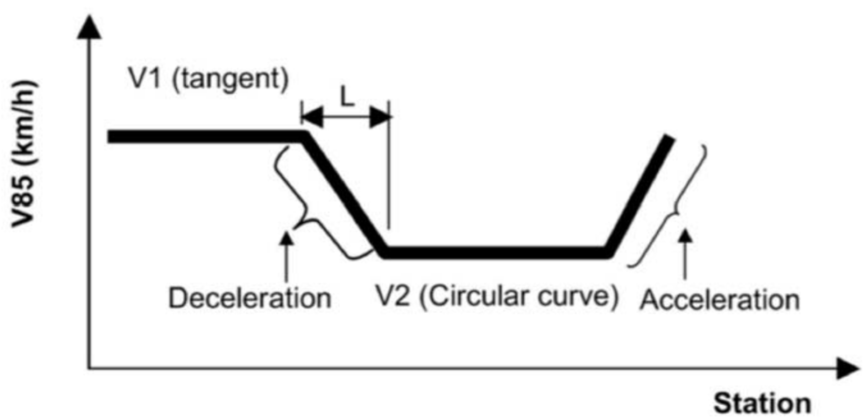


Figure 9: Speed profile (Castro et al. 2008).

Based on the speed profile shown in Figure 9, we can see that the largest speed reduction takes place at the transition point. According to the consistency-checking criterion 2 in Table 3, the large speed reduction at the transition point can make the road segment more inconsistent, which will cause safety issues. De Oña et al (2013) also found that the greater the speed reduction experienced by vehicles on a horizontal curve, the greater the number of crashes that will occur on the road section.

To select the appropriate model to check consistency and determine safety level, Camacho-Torregrosa et al. (2013) tested 15 variables; five of them proved to be the most correlated to crash rates. Then they checked the following models, which include the highly correlated variables, and calibrated the coefficients.

Table 10: Final Calibrated Models for Estimated Crash Rate (ECR) (Camacho-Torregrosa et al. 2013)

Model number	Expression	R^2
1	$ECR = \frac{1}{2.65897+(0.0570069/(R_a/\bar{v}_{85}^2))}$	39.8%
2	$ECR = \frac{1}{3.00826+(0.00056031/(R_a/\bar{v}_{85}^2))}$	42.3%
3	$ECR = e^{0.00516932} \cdot \left(\frac{R_a}{\bar{v}_{85}}\right)^{0.431919}$	35.8%
4	$ECR = \frac{1}{3.18601+(33.4686/R_a \cdot \bar{v}_{85})}$	32.8%
5	$ECR = \frac{1}{3.11287+0.0273507 \cdot (\bar{v}_{85}/\overline{\Delta v_{85}})^2}$	45.7%
6	$ECR = e^{-0.866762} \cdot e^{-0.085472 \cdot (\bar{v}_{85}/\overline{\Delta v_{85}})}$	40.1%
7	$ECR = \frac{1}{3.36708+0.0000029176 \cdot (\bar{v}_{85}^2/\overline{\Delta v_{85}})^2}$	48.2%
8	$ECR = \frac{1}{2.40939+0.00403287 \cdot (\bar{v}_{85}^2/\overline{\Delta v_{85}})}$	46.3%

where

ECR = estimated crash rates (crashes/10^6 vehicles -km)

R_a = indicator that measures area bounded by the operating speed profile and the average operating speed of each road segment, divided by the road length

$\overline{\Delta v_{85}}$ = average speed reduction, the average value of all speed reduction processes at each road segment

$\overline{v_{85}}$ = average operating speed

Based on the result of the statistical analysis, model 8 was selected because of its better performance with low-consistent road segments (though its R^2 value is a bit smaller than the R^2 value for model 7). A summary of crash rate prediction models mentioned in this report is shown in Table 11.

Table 11: Summary of Crash Rate Prediction Models

Model	Author
$relative\ risk(V) = e^{-0.822957835-0.083680149V+0.001623269V^2}$ $relative\ risk(D) = e^{0.1133374D+0.002817D^2}$	Kloeden et al. (2002)
$N_{br} = ADT \times L \times 365 \times 10^{-6} \times e^{-0.4865}$	Harwood et al. (2000)
12 ft lane width $:ACCR = -0.55 + 1.08 \cdot DC; R^2 = 0.73$ 10 ft lane width $:ACCR = -1.02 + 1.51 \cdot DC; R^2 = 0.30$	Lamm et al. (1995)
Mean accident rate = 0.05 + 0.23 * mean degree of curvature	Fink and Krammes (1995)
$Y = \exp(\beta_0)\, AADT^{\beta_1} CL^{\beta_2} \exp(\beta_3 \Delta V_{85})$	De Oña et al. (2013)
Model 1: $\ln(\text{crash rate}) = 44.323 - 25755.82/SD^2 + 93793.11/SD^4 - 8.686*10^{-3}*FPL^2 + 0.106/SD^2*FPL^2 - 1.68710^{-8}*FPL^4 + 469.071/LW^{0.5} + 44529.25/SD^2/LW^{0.5} + (1.445*10^{-2}*FPL^2/LW^{0.5} - 956.114/LW^{0.5})^2 - 93.415*SW - 660.808/SD^2*SW + 5.626*10^{-5}*FPL^2*SW + 152.084/LW^{0.5}*SW + 3.475*SW^2$, $R^2=0.9864$, AIC=-48.715 Model 2: $1/\text{crash rate}^{0.5} = 1132.667 - 3035.839/SD - 13380.54/SD^2 - 1.436*10^{-3}*FPL^2 - 4.313*10^{-2}/SD*FPL^2 + 1.752*10^{-7}*FPL^4 - 9519309/MEAN^2 - 6956803/SD/MEAN^2 - 71.254*FPL^2/MEAN^2 - 1.060174*10^9/MEAN^4 - 210.998*LW + 1963.584/SD*LW + 3.751*10^{-3}*FPL^2*LW + 3334646/MEAN^2*LW - 65.918*LW^2$, $R^2=0.9817$, AIC=18.247 Model 3: $1/\text{crash rate}^{0.5} = 23635.61 - 17107.41/SD - 12605.73/SD^2 - 1.184*FPL - 10.318/SD*FPL + 2.621*10^{-3}*FPL^2 - 25345.75*LW^{0.5} + 10829.1/SD*LW^{0.5} + 1.051*FPL*LW^{0.5} + 6744.683*LW - 156.286*SW^2 + 199.0262/SD*SW^2 - 8.073*10^{-2}*FPL*SW^2 + 89.694*LW^{0.5}*SW^2 - 2.945*SW^4$, $R^2=0.9697$, AIC=28.845 Model 4: $1/\text{crash rate} = 1331486 - 1365.183*SD - 5.771*SD^2 - 0.541*FPL^2 + 6.709*10^{-3}*SD*FPL^2 + 3.204*10^{-6}*FPL^4 - 4744279/LW^{0.5} + 2739.721*SD/LW^{0.5} + 0.873*FPL^2/LW^{0.5} + 4219166/LW + 1871.932*SW^2 - 23.51972*SD*SW^2 - 0.031*FPL^2*SW^2 - 2230.496/LW^{0.5}*SW^2 - 47.560*SW^4$, $R^2=0.9312$, AIC=171.06	Garber and Ehrhart (2000)
$ECR = \dfrac{1}{2.40939 + 0.00403287 * (\bar{v}_{85}^2/\Delta\bar{v}_{85})}$	Camacho-Torregrosa et al. (2013)
$y_{is} \mid \vec{\varepsilon}_i, \beta_s, x_{is} \sim Poisson(\lambda_{is})$	Ma et al. (2008)

Notation

AADT = average annual daily traffic (vehicles/day)

$ACCR$ = estimate of accident rate including all accidents (acc./10^6 vehicle-miles)

ADT = average daily traffic volume (vehicles/day)
AIC= Akaike's information criterion
CL = curve length (km)
D = difference in traveling speed (km/h)
DC = degree of curve (degree/100 ft) range: 0° to 25°
FPL = flow per lane (vph)
L = length of highway element (km)
LW = lane width (m)
MEAN = the mean speed (km/h)
Mean accident rate = accidents per 10^6 vehicle kilometers of travel
N_{br} =predicted number of total highway element crashes per year
R^2 = coefficient of determination
SD = standard deviation of speed (km/h)
SW = shoulder width (m)
V = free-traveling speed (km/h)
$V85$ = estimate of the operating speed, expressed by the 85th percentile speed for passenger cars (mph)
Y = estimated number of crashes in 3 years
$\beta_0, \beta_1, \beta_2$ =parameters
ECR = estimated crash rates (crashes/10^6 vehicles -km)
R_a = indicator that measures the area bounded by the operating speed profile and the average operating speed of each road segment, divided by the road length
$\overline{\Delta v_{85}}$ = average speed reduction, the average value of all speed reduction processes at each road segment
$\overline{v_{85}}$ = average speed reduction, the average value of all speed reduction processes at each road segment
$\lambda_{is} = \exp(x_{is}'\beta_s + \varepsilon_{is})$
y_{is} =crash counts for roadway segment *i*, severity level *s*
x_{is}' =severtiy level-specific explanatory variables
ε_{is} = severity-level-specific unobserved heterogeneity across roadway segments

2.8 SUMMARY

Section 2.1 showed that the relationship between operating speed and posted speed limit was nearly linear. The discrepancy between operating speed and design speed, which could cause safety issues, was also discussed. The current procedures for selecting design speed by different agencies were summarized in Section 2.2, which showed that little consideration was given to operating speed when agencies in the United States select the design speed. Some other issues related to the selection of design speed, such as the effectiveness of the old data, were also discussed. Section 2.3 explained the practice used for the selection of speed limits in Illinois. Sections 2.4 and 2.5 introduced the criteria to quantify the inconsistency in design with two strategies to narrow the gap between design speed and operating speed. It was indicated that the feedback loop method adopted by some European countries and Australia could be considered a potential solution to address speed inconsistency.

Various operating speed prediction models were presented in Section 2.6. These models have different considerations. Some of them used the geometric elements of roadways (degree of curvature, super-elevation rate, grades in the direction of travel, etc.) as independent variables while other models included the effect of different speeds (approach speed on the curve, posted speed limit, etc.).

In Section 2.7, the effects of speed, traffic flow, and roadway geometry on roadway safety were discussed. Based on their observations and the data collected, several researchers adopted statistical and mathematical methods to establish univariate and multivariate models for crash rate estimation. In addition, models based on the speed profile were established to estimate the safety of a roadway. Finally, the CMFs that consider the effect of lane width and shoulder width in crash rate prediction models were discussed.

CHAPTER 3: METHODOLOGY

A methodological framework (Figure 10) is proposed to conduct statistical and simulation analysis of the relationship between safety and design speed choices. First, a number of alternative designs are identified then checked by the AASHTO design criteria to ensure their feasibility. The operating speeds will be calculated based on the inputs and the geometric elements of these feasible designs. The difference between design speed and operating speed will be checked to determine the design consistency level. Crash rates are then predicted and checked. A benefit-cost analysis will also be conducted.

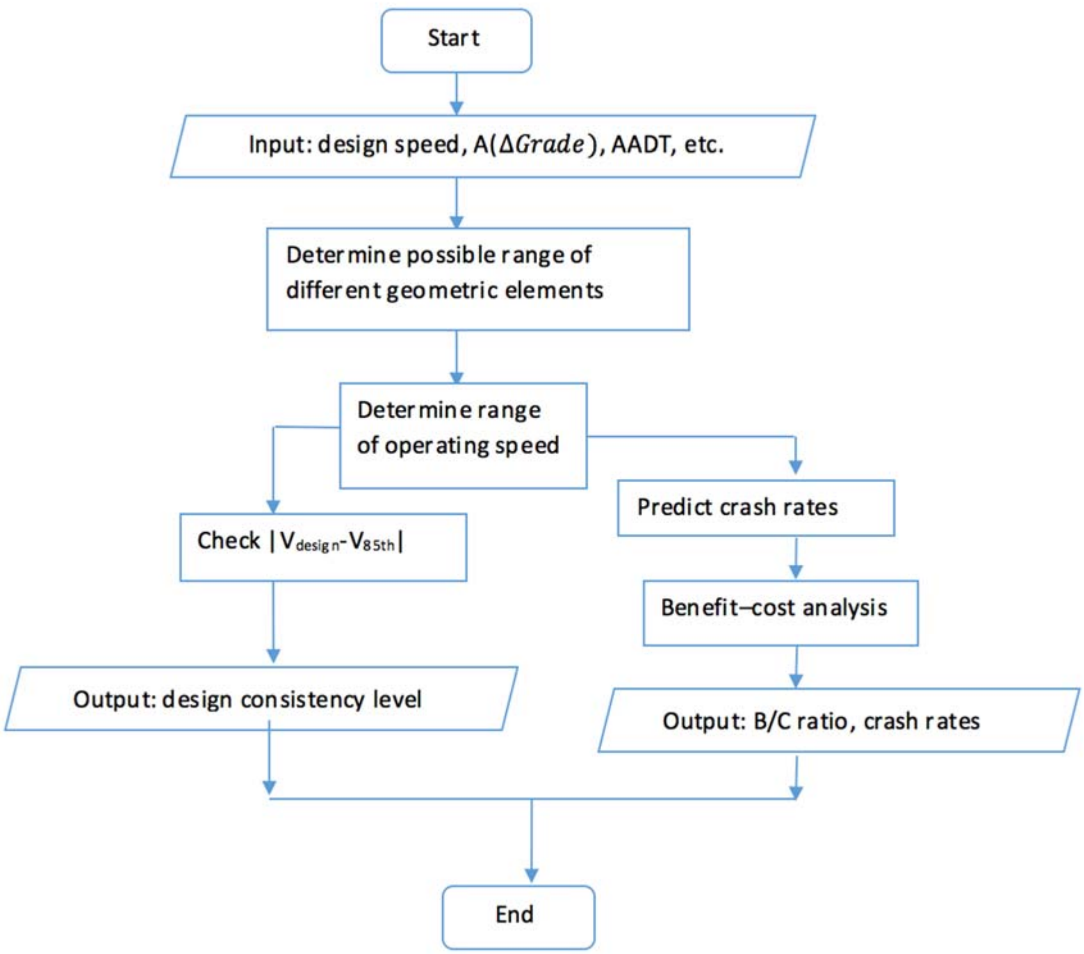


Figure 10: Flowchart to evaluate the consistency.

3.1 INTERVIEWS AND INPUTS

3.1.1 Interviews with Illinois County Engineers

To gain a better understanding of IDOT's current design guidelines and policies and identify issues of the current geometric design practices and their safety impacts on Illinois roads, we interviewed knowledgeable and experienced engineers in three counties: Sangamon, Champaign, and Menard.

The questions in our interviews cover the following topics:

- Roadway geometric design: front slope (question 2), guardrails (question 3), and clear zone (questions 6, 8, and 9)
- Design speed, operating speed, and posted speed limit: design speed and operating speed (questions 14, 16, and 18) and posted speed limit (questions 12 and 15)
- Roadway safety and improvements safety level (question 22) and improvements (questions 21 and 23)

Details of the interviews are shown in Appendix A. Analysis of the county engineers responses revealed the following:

- The discrepancy between design speed and operating speed was observed by all the county engineers interviewed (question 19). However, only design speed is used for the design of low-volume roads, though the operating speeds on these roads are very high. It is difficult to incorporate operating speed into the design procedure because agencies need to balance safety and cost (question 14). They consider posting a lower speed limit a potential strategy to narrow the gap between design speed and operating speed.
- Regarding the guardrail on low-volume roads, the agency may choose not to install the guardrails because of high maintenance cost and the restriction of the narrow right-of-way (question 3). Also, on low-volume roads, the driver could use relatively wider roads; therefore, the installation of guardrails may not be necessary (question 10).
- Hot-mix asphalt (HMA) is the most frequently used material for county roads in Illinois. Generally, the roadway surface type has little impact on the operating speed (questions 4 and 5).
- The design speed is not considered when agencies remove a fixed object from the roadside (question 6). The *Bureau of Local Roads and Streets Manual* (BLRS) is followed when culverts are installed (questions 7, 8, question 9).
- The roadway features that promote a higher operating speed are wider roads and lanes, wider shoulders, straight roads, greater radii of curves (horizontal and vertical), smoother pavement surface, etc. (question 11).
- The speed limit should be selected or altered according to the "Policy on Establishing and Posting Speed Limits on the State Highway System" (IDOT 2011) (question 15). Lower posted speed limit might be considered at the potential conflict points (school, business area) and parking areas (question 12).

- No consideration is given to operating speed when agencies select design speed (question 18).
- The feedback loop procedure used in Australia and Europe is difficult to incorporate into the design policy due to limited budgets (question 21).

3.1.2 Acceptable Range of Geometric Design

We try to consider both the horizontal and vertical roadway characteristics in our model framework. Thus, the inputs in Table 12 that characterize the roadway features in three dimensions are necessary. Also, we consider traffic characteristics (such as AADT) because they could influence crash rates and hence the selection of a clear zone.

Table 12: Summary of Inputs

Input	Description
A	Absolute value of the difference in grades $\lvert G_1\text{-}G_2 \rvert$
V	Design speed (mph)
G1	First grade in the direction of travel
G2	Second grade in the direction of travel
L_0	Horizontal distance between point of vertical intersection and the point of horizontal intersection (ft)
AADT	Annual average daily traffic (vehicles/day)
LW	Lane width (ft)
SW	Shoulder width (ft)
Sag/Crest	Sag/crest vertical curve

First, we try to obtain possible values of curve radius, super-elevation rate, and the length of the vertical curve from Table 13. Then we select the acceptable designs by testing whether the combinations of these variables meet the geometric design requirements.

Table 13: Limits of Geometric Design Data (Gibreel et al. 2001)

Geometric element (1)	Type of 3D Alignment Combination: Sag vertical curve and horizontal curve (2)	Type of 3D Alignment Combination: Crest vertical curve and horizontal curve (3)
Radius of horizontal curve, r (m)	575–900	425–1,800
Deflection angle of horizontal curve, Δ_c (degree)	5–40	5–55
Rate of superelevation, E (percent)	4.4–5.5	2.9–6.0
Length of spiral curve, L_S (m)	45–65	30–75
Grade, G (percent)	≤6.00	≤6.00
Length of vertical curve, L_V (m)	80–180	80–320
Algebraic difference in grades, A (percent)	1.15–6.00	1.00–7.10
Distance between horizontal and vertical points of intersections, L_O (m)	42.9–186.0	5.9–170.0
Shared portion of lengths of horizontal and vertical curves in same combination X (m)	≤150	≤210
Lane width, L_W (m)	3.75	3.75
Shoulder width, L_{SH} (m)	2.00	2.00

To ensure the feasibility of these combinations, we determine the stopping sight distance (SSD) first and then calculate the minimum length of vertical curvature based on SSD. For example, if we consider the sag vertical curve, the following equations are used to obtain the minimum L_v. We assume the largest L_v is the limit value in Table 13. Thus, we can obtain the acceptable range of the length of vertical curve.

$$\text{For} \quad SSD < L,\ L_m = \frac{A * SSD^2}{200(H + SSD\ \tan\beta)^2} \tag{20a}$$

$$SSD > L,\ L_m = 2SSD - \frac{200(H + SSD * \tan\beta)}{A} \tag{20b}$$

where

L_m = minmum length of vertical curve in ft

A = absolute value of the difference in grades $|G_1 - G_2|$

H = height of headlight in ft (m)

β = inclined angle of headlight beam in degree

Then the combinations of curve radius and super-elevation rate that satisfy Equation 21 and Equation 22, along with the acceptable vertical curve lengths, are considered acceptable designs in our framework.

$$R_v = \frac{V^2}{g(f_s + \frac{e}{100})} \tag{21}$$

where

$\mathrm{e} = \text{rate of roadway super-elevation (\%)}$

$\mathrm{g} = \text{graviational constant}, 32.2\ \mathrm{ft/s^2}$

$\mathrm{f_s} = \text{coefficient of side friction}$

$\mathrm{V} = \text{design speed (ft/s)}$

$\mathrm{R_V} = \text{radius defined to the vehicle's traveled path (ft)}$

$$SSD = \frac{\pi R_V}{90}\left[\cos^{-1}\left(\frac{R_V - M_S}{R_V}\right)\right] \tag{22}$$

where $\mathrm{M_s}$: middle ordinate necessary to provide adequate stopping sight distance (S) in ft (m).

3.2 OPERATING SPEED PREDICTION

The operating speeds on the curve and tangent were calculated. The speed on the curve was calculated with Gibreel's model (2001), and the speed on tangent was calculated using Zuriaga's model. Gibreel's model can be used to characterize both the vertical and the horizontal features of the road sections. According to the Gibreel's comparison, the model is more accurate than the 2D model and could fit the real data better. According to Zuriaga's analysis, the model proposed had smaller MSE and RMSE than the other models tested. Thus, it is more accurate and could better fit driver behavior.

3.2.1 Operating Speed on Curve

After the acceptable range of design variables is obtained, we used the model proposed by Gibreel et al. (2001) to predict the range of operating speeds. Equations 1a through 1j shown in Section 2.6 are used to predict the operating speeds for each design.

In each location (1 to 5, the definition of these locations can be found in Section 2.6), we predicted several operating speeds based on the designs we obtained. We assumed they all have a normal distribution and therefore used the standard errors found in Gibreel et al. (2001) as the standard deviations of the normal distribution. After normalizing the probability, we could reconstruct the distribution of the speed, which was captured by the probability density function, at each location. Once the probability density function was obtained, the mean of operating speed and the standard deviation of the speed could be calculated using Equation 23 and Equation 24, respectively.

$$v_j = \int_{-\infty}^{+\infty} v f_j(v)\, dv \tag{23}$$

$$SD_j = \sqrt{\int_{-\infty}^{+\infty} \left(v - v_j\right)^2 f_j(v) dv} \quad (24)$$

where

$j =$ index of location (1 to 5)
$f_j(v) =$ normalized probability of density function at location j
$v_j =$ mean of predicted operating speed at location j
$SD_j =$ standard deviation of predicted operating speed at location j

3.2.2 Operating Speed on Tangent

Zuriaga's model (2010) (Equation 7d) was used to predict the speed on tangent section because it proved to have better performance.

$$V_{85T} = V_{85C} + (1 - e^{-\lambda L}) \cdot (V_{des} - V_{85C})$$

where

$V_{des} =$ desired speed(km/h), assumed 110km/h from the literature
$V_{85C} =$ 85th percentile speed on previous curves obtained from the proposed model
$L =$ tangent length(m)
$R =$ curve radius(m)
$\lambda = 0.00135 + (R - 100) \cdot 7.00625 \cdot 10^{-6}$

3.3 CRASH RATE ESTIMATION

Crash rates are predicted based on the speed profile obtained in Section 3.2. It should be noted, however, that neither the speed prediction model nor the crash rate estimation model consider the effect of shoulder width and lane width. To incorporate the effects of these two features in our framework, the estimated crash rates are multiplied by the crash modification factors for these two features as given by AASHTO.

3.3.1 Crash Rate Estimation Based on Speed Profile

Camacho-Torregrosa selected five variables from 14 based on the results of correlation analysis. On the basis of the statistical analysis conducted, the following model proved to be the most appropriate due to its relatively higher R^2 and better performance for low-consistent roads segments—and therefore was used to predict crash rates.

$$ECR = \frac{1}{2.40939 + 0.00403287 * (\bar{v}_{85}^2 / \Delta\bar{v}_{85})}$$

where

ECR = estimated crash rates (crashes/10^6 vehicles -km)

$\bar{v}_{85}$ = Average operating speed

$\overline{\Delta v_{85}}$ = Average speed reduction, the average value of all speed reduction processes at each road segment

Garber's models were also tested because they incorporate shoulder width and lane width in the model directly but were found inappropriate because they cannot reflect the relationships between different entries and the crash rate directly and are sensitive to changes in geometric factors such as lane width.

3.3.2 Effect of Clear Zone, Shoulder Width, and Type on Safety

The clear zone consists of the shoulder, the recoverable and non-recoverable slopes. We assumed the same slopes for our case study and used different shoulder widths to model the effect of a clear zone.

In IDOT design, the CMFs for shoulder width and type were calculated using Equation 17:

$$CMF_{2r} = (CMF_{wra} \times CMF_{tra} - 1.0) \times p_{ra} + 1.0$$

The notations and meaning of this CMF were discussed in Section 2.7.3.

3.4 BENEFIT–COST ANALYSIS

According to NHTSA, in the decade from 2002 through 2011, speeding was a contributing factor in nearly one-third of all fatal crashes, claiming a total of 123,804 lives and resulting in an annual economic cost to society of approximately $40 billion per year. In 2011 alone, 9,944 lives were lost in speeding-related crashes.

Several studies have attempted to quantify the benefits and costs of speed limit changes on highways. The studies uniformly concluded that raising speed limits have more costs than benefits (Reed 2001) and that on rural roads in particular, there was no economic justification for increasing the speed limit.

The first step in the benefit–cost analysis process is to identify potential economic factors that may be positively (i.e., agency and non-agency benefits) or negatively (i.e., agency and non-agency costs) impacted during a typical roadway life cycle. The direct resulting costs and benefits include infrastructure impacts including roadway geometric features such as horizontal and vertical alignment and guardrail lengths, as well as other components such as speed reduction zones, fuel consumption, vehicle travel time, and road traffic crashes.

Generally speaking, infrastructure costs initially involve upgrading low-cost features, such as speed limit signs, warning signs, and tapers. For non-freeways, modifications to passing zones, signal clearance intervals, and speed reduction zones would also typically be warranted. The benefit–cost ratios associated with raising speed limits on a system-wide basis were below 1.0 for freeways and non-freeways, suggesting unfavorable economic results. This was due in large part to the substantial infrastructure costs associated with geometric modifications along certain segments that will ultimately be necessary to achieve compliance with 36 state and/or federal design speed requirements. Consequently, to avoid costly geometric improvements, it was recommended that speed limit increases on high-speed roadways should be considered only for select segments with high operating speeds, low crash occurrence, and where the increased speed limit remains compliant with design speed requirements (i.e., the new speed limit does not exceed the design speed) (Gates et al. 2015; Savolainen et al. 2014).

An economic analysis was conducted in Michigan to examine the following scenarios:

- increasing the maximum freeway truck/bus speed limit to 65 or 70 mph
- increasing rural freeway speed limits to 75 or 80 mph
- increasing existing 55 mph urban speed limits to 70 mph
- increasing speed limits on non-freeway facilities from 55 to 65 mph

This economic analysis considered road user costs and benefits, as well as installation cost for speed limit sign upgrades, and estimates of system-wide costs for necessary geometric upgrade associated with increasing posted speed limits above existing design speeds. The benefit–cost results suggest that none of the proposed speed limit policy scenarios present a favorable economic condition compared with the current policy (Gates et al. 2015).

To conduct the benefit–cost analysis and obtain the benefit–cost ratio (BCR), the following methods were used. We will improve the features at one time (for example, widening the shoulder width) and analyze the benefits and costs associated with the improvement.

3.4.1 Benefits

$$b_i = C_a \cdot R_i \cdot AADT \cdot L \cdot ECR \cdot 365 \cdot \frac{(1+s)^{n_i}-1}{s(1+s)^{n_i}} \tag{25}$$

$$ECR = ECR_{org} \times CMF_{2r} \times CMF_{ra} \tag{26}$$

$$B = \sum_{i=1}^{n} b_i \tag{27}$$

where

b_i = present value of safety benefits of countermeasure i ($)

C_a = average cost of each accident ($)

R_i = CRF (crash reduction factor) value for countermeasure i, which is the expected percent decrease in crash rates due to countermeasure implementation. Calculated as $(1 - CMF)$,

$AADT$ = average annual daily traffic (vehicles/day)

L = roadway segment length (mile)

ECR_{org} = estimated crash rates (crashes/10^6 vehicles -mile) using Model 8 in Table 10 without considering the effect of shoulder widths and lane widths

CMF_{2r} = crash modification factor for the effect of shoulder width and type on total crashes

CMF_{ra} = crash modification factor for the effect of lane

ECR = estimated crash rates (crashes/10^6 vehicles -mile) considering the effect of shoulder widths and lane widths

s = minimum attractive rate of return expressed as a decimal fraction
n_i = service life of countermeasure i
B = total benefit of improvement (\$).

3.4.2 Cost

Roadway feature upgrading costs are calculated with the following formula:

$$C = \sum_{i=1}^{n} C_i \times \Delta x_i \tag{28}$$

where

C = total cost of improvements (\$)
C_i = unit cost of the improvement in feature i (\$/unit)
Δx_i = unit of the improvement in feature i

3.4.3 Benefit–Cost Ratio

$$BCR = \frac{B}{C} \tag{29}$$

If BCR > 1, benefits exceed costs.

If BCR < 1, costs exceed benefits.

3.5 SOFTWARE DEVELOPMENT

The models developed in this project were programmed into an Excel VBA-based computer tool to facilitate decision-making. From the interface shown in Figure 11, the user could select the module to start the analysis, whereas the speed on tangent could be either predicted or entered by the user.

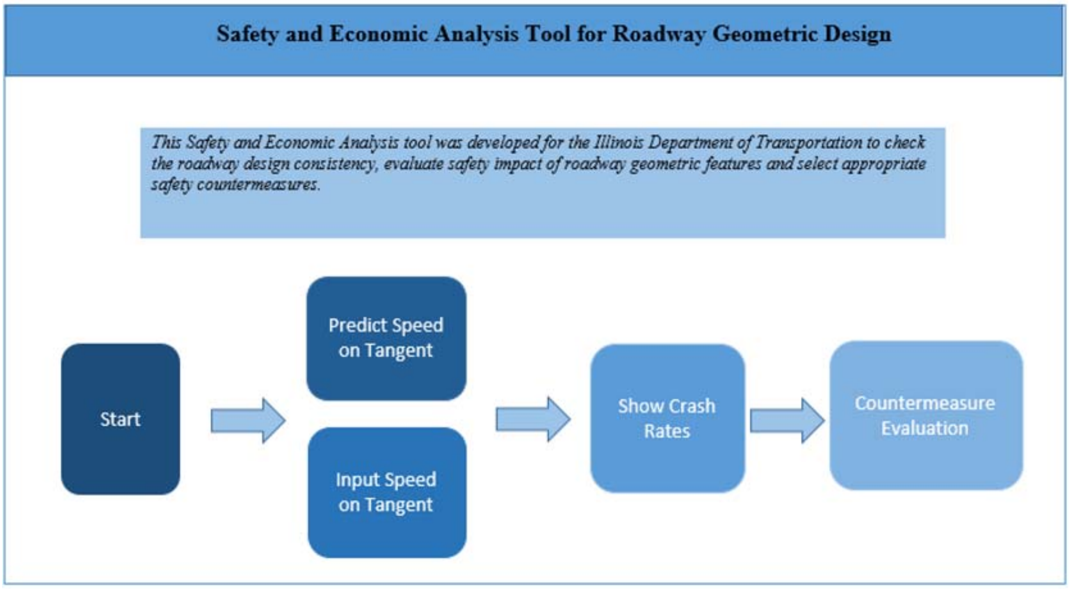
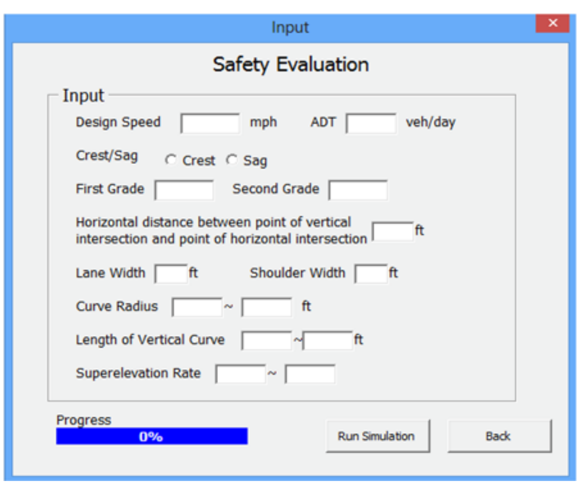
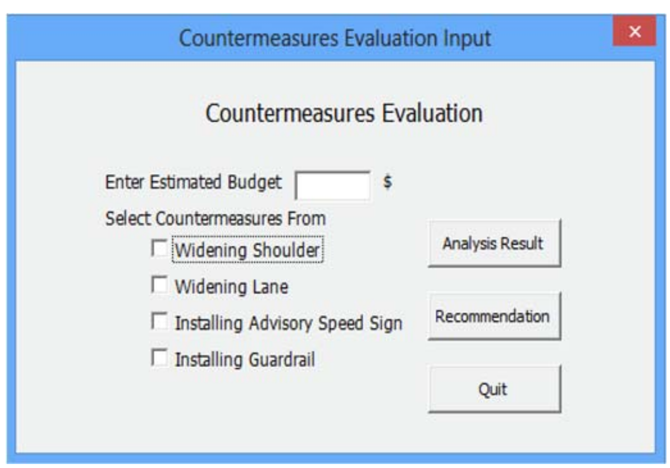


Figure 11: User Interface for Module Selection

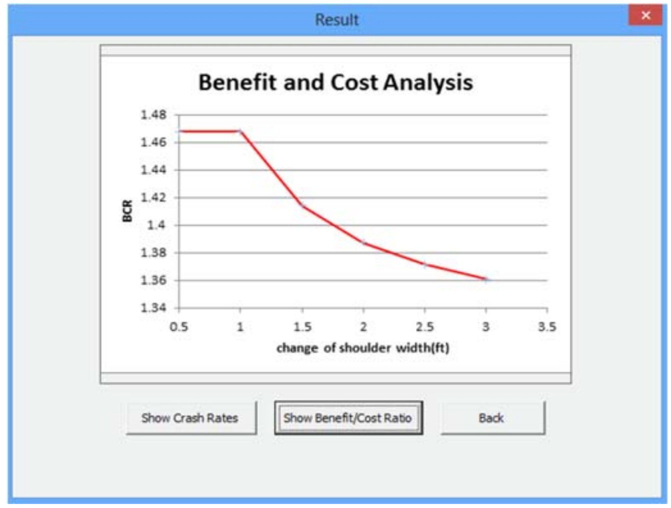
The input form for predicting operating speeds is shown in Figure 12(a). Users could enter the inputs such as traffic condition, design speed, and the basic geometric design requirements. The software could simulate the geometric design, predict operating speed distribution, and assess the expected crash rates based on these inputs. The software considered safety improvement strategies such as should widening and installing guardrails. The input user interface is shown in Figure 12(b).





(a) Input Form for Speed Prediction (b) Input form for Countermeasures Evaluation

(c) Sample Output (Crash Rates)



(d) Sample Output (Benefit-Cost Ratios)

Figure 12: User Interface for Inputs and Sample Outputs

Once the simulation is completed, the user will be able to view the results from safety evaluation and economic analysis. The results will include the expected crash rates, speed consistency level, and benefit-cost ratios for each safety improvement strategy. Figure 12(c) and Figure 12(d) show screenshots of sample outputs for widening the shoulder to different widths. The graphical user interface efficiently helps the agency visualize the safety impact and benefit/cost of countermeasures.

CHAPTER 4: CASE STUDY

4.1 INPUT INFORMATION

As an illustrative example, we consider a simple geometric design shown in Figure 13. The definition of the five locations can be found in Section 2.6. The example is a horizontal curve combined with a sag vertical curve with specified grades, shoulder width, and lane width. We used 50 mph as the design speed and assumed that AADT is 1,000 vehicles/day. A summary of the inputs involved in the example is given in Table 14.

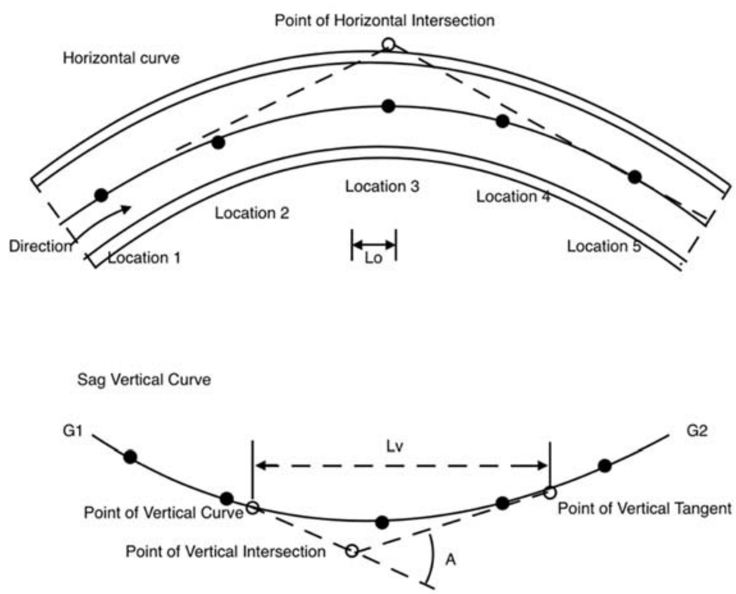


Figure 13: Combined section.

Table 14: Summary of Inputs

Input	Value	Description
A	2	Absolute value of the difference in grades $\lvert G_1–G_2 \rvert$
V	50 mph	Design speed
G1	–1	First grade in the direction of travel
G2	1	Second grade in the direction of travel
L_0	262 ft	Horizontal distance between point of vertical intersection and the point of horizontal intersection
AADT	1,000 vehicles/day	Annual average daily traffic
LW	12 ft	Lane width
SW	5 ft	Shoulder width
Sag/Crest	Sag	Sag vertical curve

4.2 PREDICTION OF OPERATING SPEEDS

First, we calculated the stopping sight distance (SSD) and then in accordance with Mannering et al. (2007), we assumed the length of the vertical curve was larger than the SSD, which is a widely used assumption in practice. The assumption was made to avoid negative minimum curve lengths. We then applied the assumption to our case and simplified Equation 20a to obtain Equation 30.

$$K = \frac{S^2}{2158} \tag{30}$$

where

$\mathrm{S} =$ stopping sight distance (ft)
$\mathrm{K} =$ rate of vertical curvature, equals L/A

Equation 30 enabled us to determine the minimum vertical length L_m, and then we could obtain several acceptable vertical curve lengths. The feasible curve radius and super-elevation rate were selected using Equation 21 and Equation 22. We used Equations 1a through 1e to model the operating speeds for each design.

$$V_S1 = 91.81 + 0.010r + 0.468\sqrt{L_V} - 0.006G_1^3 - 0.878\ln(A) - 0.826\ln(L_0)$$

$$V_S2 = 47.96 + 7.217\ln(r) + 1.534\ln(L_V) - 0.258G_1 - 0.653\mathrm{A} - 0.008L_0 + 0.020\exp(E)$$

$$V_S3 = 76.42 + 0.023\mathrm{r} + 2.300 \times 10^{-4}K^2 - 0.008\exp(A) - 1.230 \times 10^{-4}{L_0}^2 + 0.062\exp(E)$$

$$V_S4 = 82.78 + 0.011\mathrm{r} + 2.067\ln(K) - 0.361G_2 + 0.036\exp(E) - 1.091 \times 10^{-4}{L_0}^2$$

$$V_S5 = 109.45 - 1.257G_2 - 1.586\ln(L_0)$$

The meanings of the notations are shown in Section 2.6.

Applying these models to our illustrative example, we were able to reconstruct the speed distribution for different locations. Figure 14 shows the distribution of the operating speeds of our example. The maximum difference between design speed and operating speed (from the blue point to the red line) appears at location 3. This could be explained by the different weight coefficients considered in the model. Equation 1c, which was used to predict the operating speeds at location 3, includes relatively larger weighted coefficients for the curve radius and super-elevation rate. Because we had several combinations of curve radius and super-elevation rate in our case, a larger variation in operating speed could be expected at location 3.

Calculating the speed at the five locations for different design speeds achieved the results presented in Table 15, where it is seen that using lower design speeds could promise relatively lower operating speeds, which may improve safety. However, the improvement may not be significant because the strategy makes the difference between design speed and operating speed larger (as shown in Figure

15), which is not preferable (according to criterion 1 in Table 3, a difference smaller than 6.21 mph (10 km/h) indicates a good consistency level). The results indicate that the current design could somehow fail to narrow the discrepancy between design speed and operating speed when the design speed is relatively low. The speeds on tangent sections were calculated by assuming the same designs, using Equation 7d.

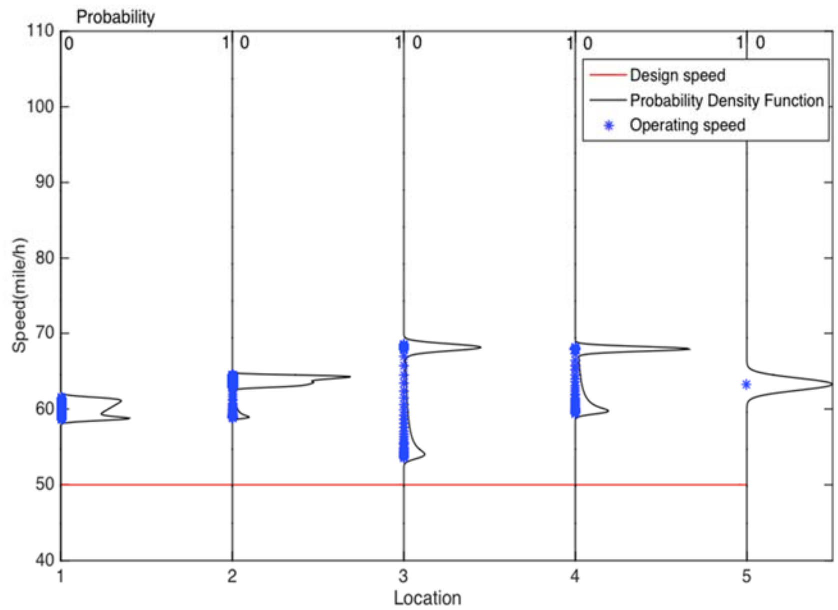


Figure 14: Operating speed prediction (design speed = 50 mph).

Table 15: Predicted v_j and SD_j

Design speed	**Point** $\boldsymbol{v_j \& SD_j}$	**1**	**2**	**3**	**4**	**5**	**Average**
60 mph	v_j/mph	61.31	65.59	65.85	66.08	63.38	64.44
	SD_j/mph	0.70	2.07	36.07	12.33	0.64	10.36
55 mph	v_j/mph	60.65	64.55	64.72	65.76	63.38	63.81
	SD_j/mph	0.83	1.90	35.31	12.08	0.64	10.15
50 mph	v_j/mph	60.12	63.38	62.98	64.97	63.38	62.97
	SD_j/mph	0.97	2.03	35.96	12.26	0.64	10.37
45 mph	v_j/mph	59.69	62.14	61.58	64.24	63.38	62.20
	SD_j/mph	1.12	2.18	36.47	12.41	0.64	10.56

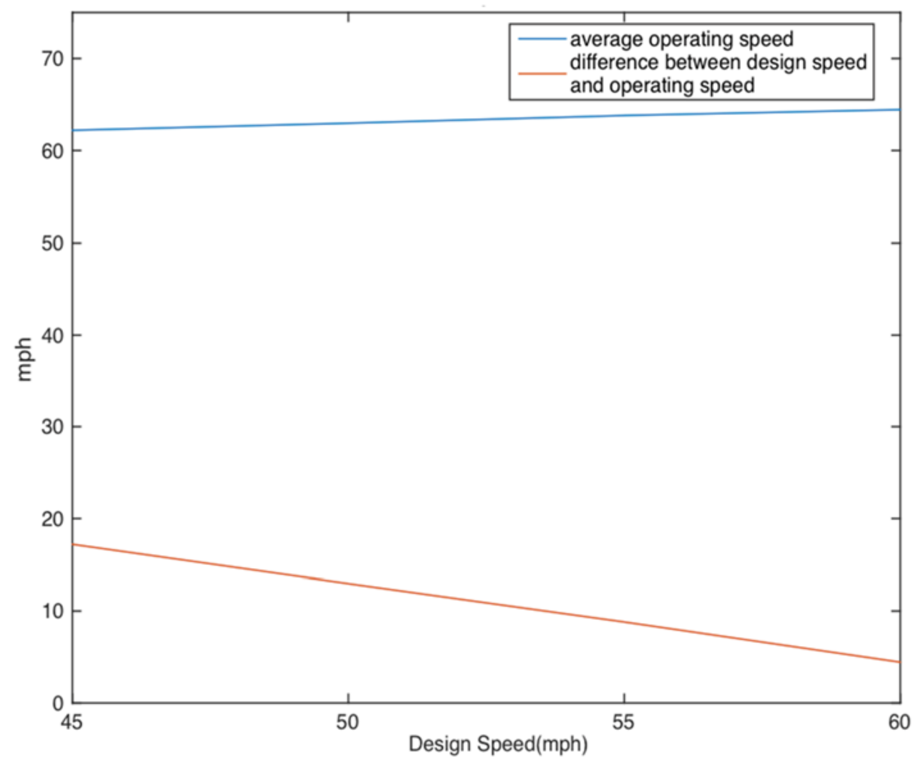


Figure 15: Operating speed prediction for different design speeds.

4.3 CRASH RATE PREDICTION

Crash rates are calculated with the following equation. In our case, we used the average speed of the five locations to represent $\bar{v}_{85}$ and the average of the speed reductions for all the designs as the $\Delta\bar{v}_{85}$. Thus, the estimated crash rate (ECR) is

$$\bar{v}_{85} = 62.6 \text{ mph}$$
$$\Delta\bar{v}_{85} = 2.031 \text{ mph}$$

$$ECR_{org} = \frac{1}{2.40939+0.00403287*\left(\frac{\bar{v}_{85}^2}{\Delta\bar{v}_{85}}\right)} = 0.1078 \text{ crashes}/10^6 \text{ vehicles -mile}$$

Then we obtain the crash rate by multiplying the ECR by the CMF_{2r} and CMF_{ra}.

$$CMF_{2r} = 1.01, CMF_{ra} = 1$$

So the crash rate for our case is

$$ECR = ECR_{org} \times CMF_{2r} \times CMF_{ra} = 0.1092 \text{ crashes}/10^6 \text{ vehicles -mile}$$

4.4 BENEFIT–COST ANALYSIS

For illustrative purposes, we widened the shoulder. Assume the shoulder was originally 5 ft wide and paved. The cost information is summarized in Table 16.

Table 16: Cost Information

Item	Cost	Source
Crash	$1,565,439/crash	Average IDOT cost for fatal (K), disabling (A), and injury (B) crashes
Shoulder Width	$4.44/ft^2	Mid-Ohio Regional Planning Commission

Therefore, we can obtain the B/C ratio (BCR) for different shoulder widths, assuming that the life cycle of the pavement is 20 years. Figure 16 shows the relationship between shoulder width and crash rates.

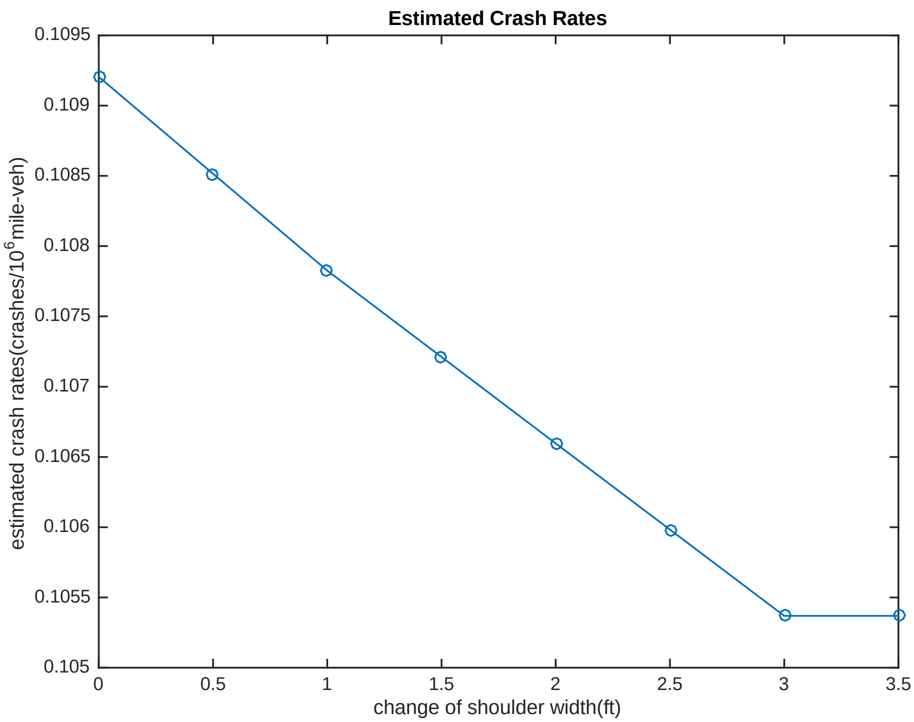


Figure 16: Relationship between shoulder width and crash rate.

When the shoulder width is smaller than 8 ft, crash rates will decrease as the shoulder becomes wider. But there is no such effect for shoulder wider than 8 ft. Figure 17 illustrates the relationship between shoulder width and BCR.

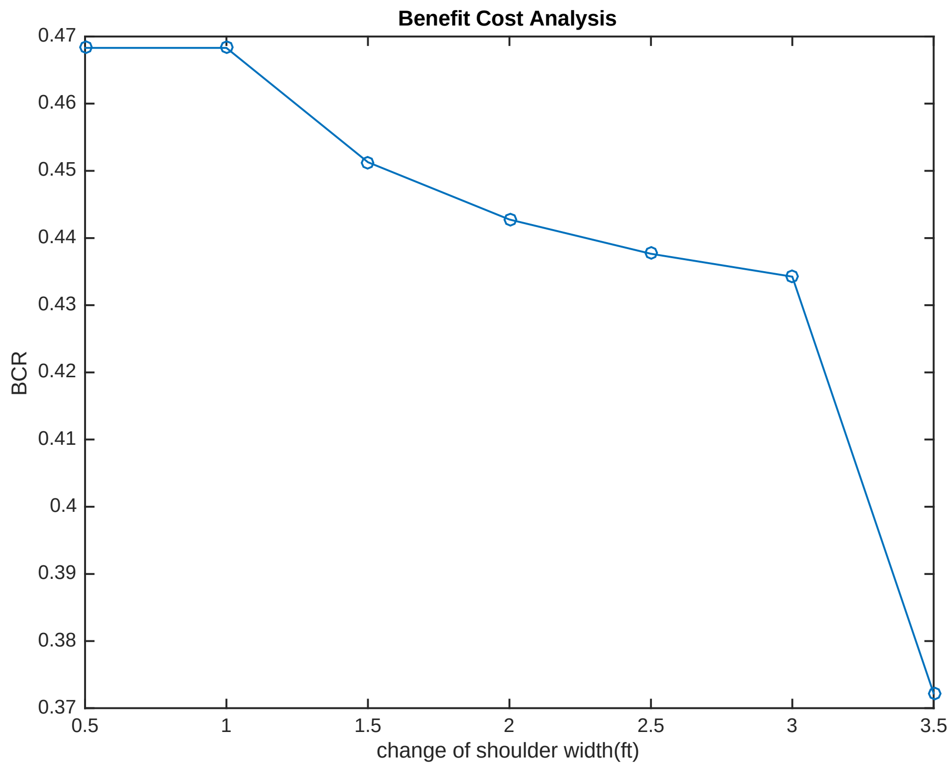


Figure 17: Relationship between shoulder width and BCR.

It can be seen that the BCRs are smaller than 1 because the savings from reducing crash rates are smaller compared with the cost. Figure 17 also shows that the BCR decreases rapidly when the shoulder width is larger than 8 ft, indicating that it is less economical to widen the shoulder to be larger than 8 ft.

4.5 CASE STUDY FOR CLEAR ZONE AND GUARDRAIL

We will conduct a case study to compare the safety effect of clear zone and guardrail. Although there are positive safety benefits to the clear zone and the guardrail, the quantitative relationships between the crash reduction and these two design elements have not been specified in the Highway Safety Manual (due to the difficulty of determining the CMFs). For this case study, we will assume the change of clear zone width is reflected by the change of shoulder width (which is a part of clear zone width). Also, we will use the CMF value of 0.93 from Elvik et al. (2009) to analyze the safety effect of installing guardrail.

For this case, we will study a hypothetical 5-mile roadway section with 4-ft paved shoulder and 12-ft clear zone. The other geometric elements are same as those in Table 14, and there is currently no guardrail. We will also assume the cost of installing guardrail is $39/ft and the other costs are the same as those in Table 16. For an ADT of 1000 vehicles, the cases below will be evaluated: the first case being the benchmark, and the other three being considered as safety improvements. The last case (#5) is an alternative approach for possible safety improvements.

- Case 1: 12-ft clear zone, 4-ft shoulder
- Case 2: 16-ft clear zone, 6-ft shoulder
- Case 3: 20-ft clear zone, 8-ft shoulder
- Case 4: Installing guardrails
- Case 5: Increasing design speed to 60 mph (~ operating speed), 20-ft clear zone, 8-ft shoulder

By applying the same method as shown in Sections 4.3 and 4.4, we could obtain the results in Table 17.

Table 17: Result of Case Study for Clear Zone and Guardrail

Case	CMF	Crash Rate (Crashes/10^6 mile-veh)	Benefit $	Cost $	Benefit/Cost
1	1.026	0.11			
2	1	0.107	106,448	227,308	0.468
3	0.977	0.105	201,284	454,615	0.443
4	0.93	0.997	397,803	1,029,600	0.389
5	0.977	0.11	0	> 454,615	0

We could observe that installing the guardrail (case 4) could reduce more crashes than widening the shoulder and clear zone (cases 2-3). However, the cost is also higher and the benefit/cost ratio is smaller. This implies that it might not be cost-effective to install guardrail for this site. Among the two strategies on widening shoulder/clear zone, case 2 slightly outperforms case 3, but neither seem to provide a benefit/cost ratio larger than 1. In such case, the safety improvement benefits do not seem to justify the respective costs. It should also be noted that the result for guardrail should be examined more carefully since its calculation depends on a CMF from the literature (which has a larger standard error). In case 5, we increase the design speed and there will be a corresponding increase in the operating speed, speed reduction. If the shoulder is not widened, the expected crash rates would have been 0.1134 crashes/106 mile-veh, which is undesirable. When the shoulder and clear zone are widened, the crash rate would be 0.1134*0.977=0.11 crashes/106 mile-veh, which means the safety improvements were not able to offset the effect of increase in crash rates due to higher speed.

CHAPTER 5: SUMMARY, FINDINGS, AND RECOMMENDATIONS

5.1 SUMMARY

To quantify the discrepancy between design speed and operating speed as well as the speed inconsistency on successive road segments, this study presented a methodological framework to evaluate the safety and consistency levels of design in Illinois. Interviews with county engineers and an extensive literature review of operating speed and crash rate prediction models, which served as the base of the framework, were conducted. A benefit–cost analysis tool was developed and used for the economic evaluation of roadway elements improvements.

5.2 MAIN FINDINGS

Based on the analyses and results, the main findings of this study may be summarized as follows:

- There exists a discrepancy between design speed and operating speed, which discrepancy causes safety problems.
- The relationship between operating speed and posted speed limit is relatively linear.
- Current IDOT policies about design speed selection are based primarily on the functional classification and give little consideration to operating speed.
- The speed limits in Illinois are selected based on roadway types and vehicle types and could be altered based on the results of speed studies. Various approaches for setting speed limits are used in other states and countries, which differ from those used in Illinois. Some of these approaches are based on crash types and computer simulation.
- There are three criteria for roadway design consistency checking: checking the difference between design speed and operating speed, checking the difference between operating speed on successive roadway sections, and checking the adequacy of side friction.
- The solution for narrowing the discrepancy between design speed and operating speed could be through altering the design speed by adding 5 or 10 mph to the speed limit and the feedback loop method. However, the former option might not be sufficient for some rural roads, and the latter may lead to a relatively high design speed.
- Using higher design speeds to determine the roadway safety features could potentially increase the roadway safety level. However, the driver might also tend to drive faster on such roads and the crash rates could also be higher. Hence, increasing design speed should be treated carefully.
- The operating speed on tangents is associated with curve radius, tangent length, and the operating speed on the previous curve.
- The crash rate is associated with various factors including operating speed, speed deviation, speed reduction, geometric elements, traffic volume, and roadway type.

- The safety impact of shoulder width is associated with traffic volume and shoulder type. Generally, widening the shoulder beyond 8 ft wide would not improve safety and thus is not economical. AASHTO recommends the use of crash modification factors to reflect the safety impact of shoulder width.
- Current design practice in IDOT is not able to narrow the discrepancy between design speed and operating speed when the design speed is relatively low.

5.3 RECOMMENDATIONS AND FUTURE RESEARCH

The following recommendations are made:

- The discrepancy between design speed and operating speed should be checked when the design speed is relatively low, which is common for rural roads.
- Detailed engineering, economic, and safety analyses should be performed prior to increasing the design speed, and increasing design speed should be considered for roadway sections where speed limit compliance is maintained after the increase (to avoid costly geometric improvements).
- The speed reduction on the tangent-to-curve transition could influence the roadway safety and should also be checked.
- Economy is an important factor that should be given careful consideration if the shoulder is to be widened to more than 8 ft.

In the future, the following research could be conducted:

- Incorporate the difference between design speed and operating speed in crash rate prediction models.
- Compare different operating speed prediction models and crash rate prediction models if data are available.
- Find the relationship between speed, severity of accidents, and geometric elements.

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APPENDIX A: INTERVIEWS WITH IDOT ENGINEERS

SUMMARY OF INTERVIEWS

Date	Department	Representative	Title	Interviewer
24-Mar-16	Champaign County Highway Department	Mr. Jeff Blue	County Engineer	Yanfeng Ouyang, Khair Jadaan, Zhoutong Jiang
5-Apr-16	Sangamon County Highway Department	Mr. Brian Davis	Assistant County Engineer	
		Mr. Brian Wright	Planning Engineer	
7-Apr-16	Menard County Highway Department	Mr. Tom Casson	County Engineer	

QUESTIONS AND ANSWERS

1. Do you maintain your Agency's roads differently for roads with low volume and low operating speeds than with high operating speeds? (Low volume roads are considered as roads with a traffic volume of 400 vehicles per day or less).

   <u>Champaign County Highway Department Response</u>: Almost all agencies will treat these roads differently because of the lack of funding. Especially in Champaign County, there are 1600 miles of township roads and it would be too expensive to treat these roads the same way as the high speed roads. Also, in IDOT practice, 55mph is generally used as the Statutory Speed for low volume roads.

   <u>Sangamon County Highway Department Response</u>: They are treated the same. No preference will be given to the roads based on ADT.

   <u>Menard County Highway Department Response</u>: Yes, county highways are maintained better than rural roads

2. What is your Agency's maximum allowable front slope on low volume roads with high operating speeds? (The operating speed is considered as the 85th percentile speed of free flowing traffic).

   <u>Champaign County Highway Department Response</u>: 3:1 is the maximum allowable front slope on low volume roads and sometimes 4:1 is used. Also, 2:1 could be used for the maximum back slope.

Sangamon County Highway Department Response: 3:1 is chosen as the maximum allowable front slope. 2:1 can be considered if necessary.

Menard County Highway Department Response: For reconstruction, 4:1 is chosen as the maximum allowable front slope. 3:1 can be considered for deep areas. The back slope is same depending on how deep.

3. What is your Agency's policy for installing guardrail on your low volume roads?

   Champaign County Highway Department Response: In terms of installing guardrails in Illinois, the *Bureau of Local Roads and Streets Manual* should be referred to. In Champaign County practice, we may try to avoid installing guardrails due to high maintenance cost.

   Sangamon County Highway Department Response: We follow the IDOT guideline.

   Menard County Highway Department Response: Sometimes, the right of way is too narrow (40ft to 60ft) to enable the installation of guardrails even if we want to. Actually, the 4-ft-wide shoulder is needed to install the guardrail but there are some 2-ft-wide shoulders in our roads, which is too narrow to allow the installation of guardrails.

4. What is your Agency's policy for which type of roadway surface you typically install for low volume road projects? Do you factor in the roadway surface type on roads with a higher operating speed?

   Champaign County Highway Department Response: HMA pavements are constructed for almost all the roads; concrete pavements are seldom used. The seal coat and chip seal are commonly used for township road treatment because they are easier to be constructed with less equipment.

   Sangamon County Highway Department Response: We only maintain the county roads, while the township agencies maintain their own roads. Based on the benefit-cost analysis and the IDOT policy, no concrete pavement will be constructed due to difficulty in maintenance. The seal coat and hot mix asphalt are widely used.

   Menard County Highway Department Response: In township roads, the chip seal surface is commonly used. In the county highways, the cold mix asphalt will be used. For county road, hot mix asphalt will be used.

5. In your opinion what effect does the roadway surface type have on the operating speed?

   Champaign County Highway Department Response: Many roads are tilted so the crown (rounded) effect could slow the drivers down. Roadway roughness and surface type have an insignificant effect on the operating speed.

Sangamon County Highway Department Response: The roughness could have a small impact on the operating speed by influencing the driver's feeling on the road.

Menard County Highway Department Response: Not much, if some materials on the surface are lost, it will have influence.

6. When you remove a fixed object (such as trees or brush) from near the roadway, do you consider the design speed or operating speed? Do you remove trees/brush for a greater distance from the roadway for higher operating speeds?

   Champaign County Highway Department Response: The operating speed is considered while the design speed is often not known. On high speed roads, IDOT will make sure these objects are removed. However, on some township roads, it depends on the safety condition and the budget.

   Sangamon County Highway Department Response: The operating speed is considered while the design speed isn't. Based on the *Bureau of Local Roads and Streets Manual (BLRS)*, we clear up to the right of way rather than clear zone.

   Menard County Highway Department Response: We don't do calculations based on these two speeds. We just clear the right of way, especially if it is narrow.

7. When you install a new culvert on a low volume road, is it your Agency's policy to extend it beyond the clear zone or protect it with a barrier?

   Champaign County Highway Department Response: The new culvert will not always be extended to clear zone for township roads. It would be difficult to implement these in township roads because of a limited budget. Elected highway commissioners may not approve such extensions if conflicts with other land use needs exist.

   Sangamon County Highway Department Response: Yes, we follow the *Bureau of Local Roads and Streets Manual (BLRS)* rules.

   Menard County Highway Department Response: Yes, we try to extend it as far as possible. We will make the slope flatter and enable the longer culverts.

8. What is your opinion regarding the clear zone requirements for culverts on a low volume, high operating speed roadway? Do you think a 6 or 7-foot clear zone policy is sufficient for these types of roadways?

   Champaign County Highway Department Response: The answer should be site specific. Generally, if it is a new project, then 6-7 feet may not be sufficient. But if only the width of ditch and shoulder is considered, then it might be fine. If it is a maintenance project, then it could be too costly to widen the clear zone.

   Sangamon County Highway Department Response: We follow the Bureau of Local Roads and Streets Manual (BLRS) rules.

Menard County Highway Department Response: The 6 or 7-foot clear zone may not be sufficient, but without enough right of way, we are unable to extend it.

9. Does the culvert or structure height enter into your decision to widen the structure to outside the policy clear zone or provide guardrail, or do you believe following the current clear zone policy is sufficient?

   Champaign County Highway Department Response: Yes, the height is a deciding factor. The higher the structure, the more concerns it will raise.

   Sangamon County Highway Department Response: We follow the *Bureau of Local Roads and Streets Manual(BLRS)* rules and believe it is sufficient since few accident cases related to the clear zone widths are observed.

   Menard County Highway Department Response: Yes. We try to have sufficient clear zone but it may not be possible.

10. What is your opinion regarding requiring the installation of roadway approach guardrail at all new bridges on low volume roads?

    Champaign County Highway Department Response: Installing guardrail near a bridge is costly; e.g., approximately 10,000 dollars per bridge. Installing guardrails could be difficult also because the cost of removing farm entrances near the bridges could be relatively high. Sometimes it is not necessary to add guardrail if the bridge is already wide enough.

    Sangamon County Highway Department Response: Actually, the guardrail can be a hazard by itself. Also, it has high maintenance costs. Especially in the low-volume road, drivers could use wider areas so the guardrails may not be necessary.

    Menard County Highway Department Response: At bridges on low volume roads, the guardrails are installed at the approach while they are not installed at the departure. But on county highways, there are guardrails at both the approach and the departure.

11. What are some of the roadway features that promote a higher operating speed?

    Champaign County Highway Department Response: Widening the shoulder width and straightening the roads will promote a higher operating speed. On the contrary, putting strips on roads, especially near intersections, could "psychologically" narrow the perceived lane width and make people slow down.

    Sangamon County Highway Department Response: Wider roads, wider shoulders, the greater radius of vertical and horizontal curves could promote a higher operating speed. The surface of the roadway may have a smaller influence also. Some township roads may have poor surface materials (aggregate), which could influence the operating speed.

Menard County Highway Department Response: Wider roads, wider shoulders and smoother pavement surface.

12. In your opinion, what are some roadway features for which you would be influenced to consider posting a lower speed limit?

    Champaign County Highway Department Response: Number of crossings in roadways and the potential conflict points (schools, business) should be considered when posting a lower speed limit. Meanwhile, most roadside hazard is not considered.

    Sangamon County Highway Department Response: The lower speed limit could be posted at roadways that are adjacent to parking areas, pedestrian crossings and the business access points.

    Menard County Highway Department Response: Sight distances need to be considered when posting speed limits. Speed limits should be posted at curves (horizontal and vertical) and residential areas.

13. What is your Agency's policy on the installation of advisory speed and/or curve signs along a low volume roadway?

    Champaign County Highway Department Response: The decision is based on driving experiments with the accelerometer.

    Sangamon County Highway Department Response: The AASHTO manual should be referred to when we install the advisory speed limit signs. The MUTCD manual is used when the curve signs are installed.

    Menard County Highway Department Response: More advanced warning signs are installed on sharp curves.

14. What is your Agency's policy regarding the use of the design speed for the design of low volume roads when you know the operating speeds are much higher?

    Champaign County Highway Department Response: Only design speed is used because there are too many factors that limit the incorporation of operating speed. Also, the agency has to balance between safety and the cost.

    Sangamon County Highway Department Response: We will try to design on speeds that are close to the posted speed limits. We may also post a lower speed. Also, the higher volume could decrease the speed.

    Menard County Highway Department Response: There are few new roads, we mainly construct new bridges. Since the ADT is pretty low, it doesn't matter so much even the operating speeds are much higher. Also, the cost is another factor we have to consider.

15. Does your Agency have a policy regarding when a posted speed limit on a low volume road is changed? This includes locations where there is no existing posted speed, and a speed limit less than 55 mph is proposed.

    Champaign County Highway Department Response: A speed study that measures the speeds in 24 hours will be conducted. The study will focus on the potential conflict points with nearby schools and business areas.

    Sangamon County Highway Department Response: Yes, we strictly follow the *Policy on Establishing and Posting Speed Limits on the State Highway System (IDOT 2011).*

    Menard County Highway Department Response: There is no policy to change the posted speed limit on the low volume road. We will follow the *Policy on Establishing and Posting Speed Limits on the State Highway System.* If speed reduction is needed, we will do speed study and find out whether the speed limit is enough.

16. If your Agency could change the policy regarding design speeds vs operating speeds, what would you recommend?

    Champaign County Highway Department Response: The *Bureau of Local Roads and Streets Manual* could be updated.

    Sangamon County Highway Department Response: Since most roads maintained by our agency is two-lane roads, we could improve safety at a low cost by enforcing the speed limit.

    Menard County Highway Department Response: It depends on the roads. Actually, in some successive roadway sections, different design speeds are used, which could be problematic. Also, the modern cars could make the drivers' overestimate the conditions of the roads, which should be considered in the policy.

17. What types of speed are encountered in the design of low volume roads, and how are they related and implemented?

    Champaign County Highway Department Response: No, only design speed is considered.

    Sangamon County Highway Department Response: The design speed and posted speed limit are encountered in the design of low volume roads. Generally, the design speed is lower than the posted speed limit. The right of way cost and the volume are considered when implementing these speeds.

    Menard County Highway Department Response: 55mph is used as the design speed. For financial reasons, 45mph is used for design speed and posted speed limit in some areas.

18. What method is used by your Agency to determine the design speed? Does it give even a minor consideration to the operating speed?

Champaign County Highway Department Response: The functional classification in *Bureau of Local Roads and Streets Manual(BLRS)* should be used to select the design speed. Little consideration is given to the operating speed.

Sangamon County Highway Department Response: The *Bureau of Local Roads and Streets Manual (BLRS)* is used. It doesn't consider operating speed.

Menard County Highway Department Response: The *Bureau of Local Roads and Streets Manual(BLRS)* is used.

19. On two lane rural highway alignments, is there empirical evidence of disparities between design speeds and operating speeds?

    Champaign County Highway Department Response: Yes, the operating speed may be 5- 10 mph greater.

    Sangamon County Highway Department Response: Based on our experience, the answer should be yes.

    Menard County Highway Department Response: Yes, the operating speed is larger than design speed and the statutory speed limits.

20. Based on your practical experience, how would you professionally, comment on the Illinois policy on selecting and applying design speed?

    Champaign County Highway Department Response: Benefit and cost should be considered if any changes towards the design policy are considered.

    Sangamon County Highway Department Response: The current policy is fine. It gives consideration to the classification of rural and urban, the 2-lane and 4-lane scenarios.

    Menard County Highway Department Response: Increasing the design speed might not be a good choice to narrow the discrepancy between design speeds and operating speeds for typical counties. But for other bigger counties, it may be a good choice because the operating speed is relatively high.

21. Would you consider the modifications to the design-speed concept in Australia and Europe in a possible revision to U.S. policy on design speed?

    Champaign County Highway Department Response: It might be beneficial but difficult to be incorporated into the policy due to the limited budgets.

    Sangamon County Highway Department Response: It may be good for interstate highway but not efficient for local roads. We also need to consider the cost.

    Menard County Highway Department Response: We don't have so many new roads. Even if we design on higher speed, we still control the speed by statutory speed.

22. How do you judge the level of road safety using the current practice?

    Champaign County Highway Department Response: The crash reports are used.

    Sangamon County Highway Department Response: We have methods to adjust the level of road safety while some factors such as the human factors and the animal factors are out of our engineering control.

    Menard County Highway Department Response: The current level of road safety is fine.

23. What would you recommend to improve the safety level?

    Champaign County Highway Department Response: (1) Most fatalities on rural roads are caused by vehicles hitting fixed objects (e.g. power poles, mailboxes). So removing these fixed objects, if possible, and clearing the roads will improve the roadway safety. (2) Proper signage will improve safety. (3) Proper maintenance could improve safety.

    Sangamon County Highway Department Response: The speed enforcement could be used to improve the safety level. Also, the zero tolerance of alcohol may be helpful.

    Menard County Highway Department Response: Some countermeasures that are not related to speed should be considered, such as adding the advisory speed on curves and adding chevrons.

24. Does your Agency consider the transition from approach tangents to horizontal curves and the speed reduction in design?

    Sangamon County Highway Department Response: The advisory speed signs and the warning signs are located at appropriate locations to allow for speed reduction.

    Menard County Highway Department Response: The curve warning signs and advisory speed signs will be used.

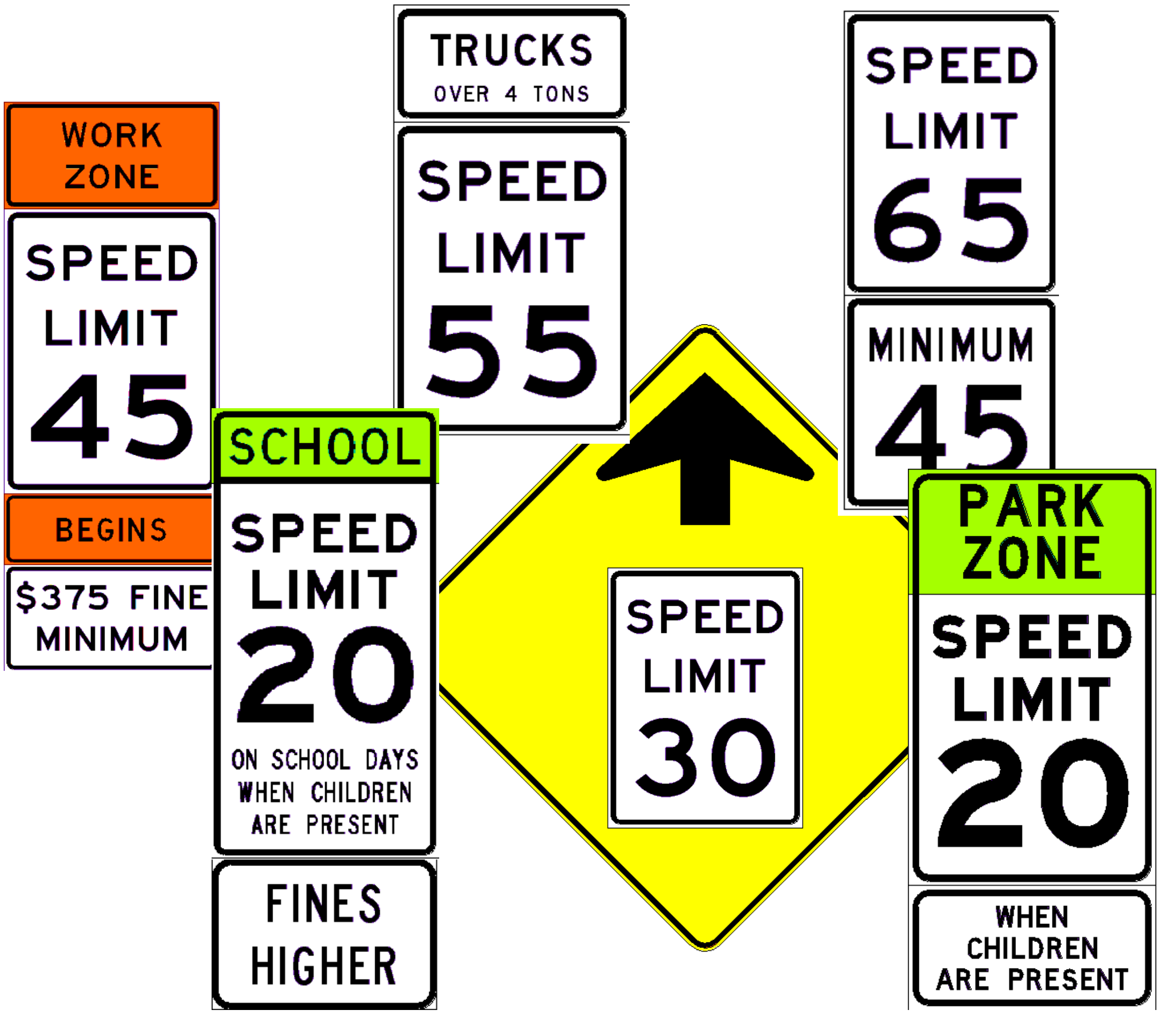
25. Are there any other aspects of this issue that you feel should be considered?

    Champaign County Highway Department Response: A model that could quantify the relationship between the operating speed and the design speed would be helpful.

    Sangamon County Highway Department Response: We actually have some helpful examples in practice to share. For the reverse curve, we could use sensors to collect the speeds of drivers and give feedback to them so the proper operating speed could be chosen. For the 2-way stop signs, the flash lights at the stop sign can be used as the feedback to give driver guidance.

    Menard County Highway Department Response: Putting rumbles strips on the road in order to "psychologically" narrow the lane width and slow people down may not be a good choice on the hot mix asphalt road. Also, if enough funds are available, the reduction of lane widths may be considered

APPENDIX B: IDOT POLICY ON ESTABLISHING AND POSTING SPEED LIMITS



Policy on Establishing and Posting Speed Limits on the State Highway System

March 2011

Illinois Department of Transportation

POLICY ON ESTABLISHING AND POSTING SPEED LIMITS ON THE STATE HIGHWAY SYSTEM

ILLINOIS DEPARTMENT OF TRANSPORTATION – BUREAU OF OPERATIONS

APPLICATION OF POLICY TO CITIES, COUNTIES AND OTHER LOCAL AGENCIES

The Illinois Vehicle Code does not require local agencies to obtain department approval for speed zones on roads under their respective jurisdictions. While the procedures contained in this policy may be used for altering speed limits on any public highway, use of such procedures by local agencies is not required by statute. If a local agency wishes to ask a district for review of a speed zone, the district may, of course, do so. However, when responding back to the agency, a statement should be included indicating that the comments are not to be considered as either approval or disapproval. Local Agencies should refer to Section 11-604 of the Illinois Vehicle Code for additional information and specific regulations regarding the alteration of speed limits on local roads.

GENERAL SPEED LIMITS

Speed limits on highways under the jurisdiction of the department shall be established on the basis of the latest revisions/editions to Article VI of the Illinois Vehicle Code (IVC), the Illinois Manual on Uniform Traffic Control Devices (IMUTCD), the Standard Specifications for Road and Bridge Construction, the Highway Standards and this policy. Night speed limits shall not be used.

A. <u>Statutory Speed Limits</u>

Section 11-601 of the IVC spells out the statutory speed limits in effect in Illinois. These limits may be enforced without any signing.

Outside Urban Districts

Freeways/Expressways

This category is defined as highways designated by the department which have at least 4 lanes of traffic where the traffic moving in opposite directions is separated by a strip of ground which is not surfaced or suitable for vehicle traffic. For the purposes of this policy, this includes all full freeways (Interstate and interstate-type freeways).

Passenger cars, buses, and trucks with gross weights of 4 tons or less	65 mph
Vehicles towing trailers, housecars, and campers	65 mph
Trucks with gross weights of over 4 tons	
(Interstate Routes)	65 mph
(All Non-Interstate Routes)	55 mph
(Within Cook, Dupage, Kane, Lake, McHenry, and Will Counties)	55 mph

This also allows the department to apply these limits to designated sections of rural expressways with full control of access and at-grade intersections rather than interchanges. In general, this should only be done where engineering judgment indicates such limits may be safely accommodated. Short sections should be avoided.

Conventional Highways

All vehicle types	55 mph

Inside Urban Districts (All vehicle types)

All streets and highways	30 mph
Alleys	15 mph

"Urban District" is defined in Section 1-214 of the IVC as "The territory contiguous to and including any street which is built up with structures devoted to business, industry or dwelling houses situated at intervals of less than 100 feet for a distance of a quarter of a mile or more." *Note that whether the street or highway in question is inside or outside of the corporate limits of a community is not included in this definition and therefore, is not applicable to the determination of where such statutory speed applies. This means that the statutory speed on an unposted street within the corporate limits of a community but outside an urban district would be 55 miles per hour.*

B. <u>Altered Speed Limits</u>

State statutes allow the department to alter certain of the statutory speeds either up or down (statutory speeds of 55 and 65 miles per hour may only be altered downward). State statutes and the Illinois Manual on Uniform Traffic Control Devices require that such altered speed limits be based on "… an engineering study that has been performed in accordance with traffic engineering practices. The engineering study shall include an analysis of the current speed distribution of free-flowing vehicles."

The following investigation and selection criteria shall be used to determine altered speed limits on streets and highways under the jurisdiction of the department. While it is not mandatory that local agencies use this format and criteria, it is recommended. Regardless of the form the engineering and traffic investigation takes, it should be based on valid traffic engineering principals, an analysis of the speed distribution of free-flowing vehicles, and be well documented.

Perceived speed enforcement tolerances shall not be taken into account in the setting of speed limits.

Prevailing Speed

The determination of the prevailing speed of free-flowing traffic is the basic step in establishing an altered speed limit either lower or higher than the statutory limit (statutory speeds of 55 and 65 miles per hour may only be altered downward). This is based on the nationally accepted premise that a majority of the drivers will drive at a speed which they judge to be safe and proper. The prevailing speed is the computed average of the following three sets of data, measured during free-flowing traffic conditions:

1. EIGHTY-FIFTH PERCENTILE SPEED: The 85th percentile speed is defined as the speed at or below which 85 percent of the vehicles are traveling. This speed is determined on the basis of spot speed studies, normally made with a concealed radar or laser speed meter.

Spot speed studies should be made as close as practical to the center of the zone which is being studied. If the zone is in excess of one mile in length in rural areas or 1/2 mile in urban areas, studies should be made at two or more locations. Care must be exercised to be sure that the data are collected in such manner and at such times that they are a true indication of normal conditions. Such conditions normally prevail under good weather conditions, on dry pavement, during daylight hours, outside of rush periods, and on any day except weekends or holidays. Observations should not be made immediately following a crash, when traffic is influence by construction or maintenance operations, or during a period of greater than normal enforcement. Every effort should be made to conceal the fact that speeds are being recorded.

Speeds should be observed for at least 100 passenger cars/vans and pickup trucks in each lane in each direction. Speeds of vehicles over four tons in size should not be used in determining altered speed zones. On lower-volume roads where it would be difficult to sample 100 vehicles in each direction, the study may be terminated after three hours. When traffic is travelling in platoons, the speed of the lead vehicle(s) should be used. Following vehicles tend to base their speeds on the lead vehicle. Use of following vehicles will tend to bias the recorded speeds downward. Care should also be taken to avoid recording the speeds of a disproportionate number of high speed vehicles to avoid an upward speed bias.

2. UPPER LIMIT OF THE 10 MILES PER HOUR PACE: The 10 mph pace is defined as the 10 mph range containing the most vehicles. This is determined on the basis of the spot speed studies discussed above.

3. AVERAGE TEST RUN SPEED: Average test run speeds are determined on the basis of five vehicle runs in each direction over the length of the proposed zone. It is not necessary to use an unmarked vehicle, however the use of any vehicle which might be mistaken for a law enforcement vehicle should be avoided. Observations should be made under the same general conditions noted above for spot speed studies. The prime consideration in use of test runs is to approximate the median speed. To accomplish this, the driver should try to "float" in the traffic stream. On multi-lane roads, the driver should pass as many vehicles as pass the test car. Use of test run speed is optional on lower-volume roads and should not be included when determining the prevailing speed for very short zones or for any specific type of vehicle other than passenger cars/vans.

The prevailing speed, to the nearest 5 miles per hour, may be used directly as the Altered Speed Limit, subject to any further adjustment resulting from reviewing the Anticipated Violation Rate as set forth below. However, in certain cases, a lower altered speed limit may be justified on the basis of supplementary investigations.

Optional Supplementary Investigations

The selected Altered Speed Limit may differ from the established prevailing speed (not the proposed posted speed) by up to 9 miles per hour when justified by further investigation. Such investigations shall be limited to studying any or all of the following four conditions:

1. HIGH-CRASH LOCATIONS: If the zone being studied contains a portion of a high-crash segment or contains a high-crash intersection as shown on the most recent 5% report as distributed by the Bureau of Safety Engineering, the prevailing speed may be reduced by 10%.

2. ACCESS CONTROL: The effect of driveways and other entrances is determined by using an "access conflict number." For this purpose, field entrances or driveways to single-family dwellings shall have a conflict number of 1. Minor commercial entrances and driveways serving multi-family residential units and minor street intersections shall have a conflict number of 5. Major commercial entrances, driveways serving large multi-family developments and major street intersections shall have a conflict number of 10. If the total access conflict number within a proposed zone exceeds those shown in the following table, the prevailing speed may be reduced by the percentages indicated.

Access Conflicts Per Mile	Percent Reduction in Speed
40 or less	0
41 - 60	5
61 or more	10

3. PEDESTRIAN ACTIVITY: Where no sidewalks are provided or where sidewalks are located immediately behind the curb and the total pedestrian traffic exceeds ten per hour for any three hours within any eight-hour period, the prevailing speed may be reduced by 5 percent. Pedestrians crossing the route at intersections or established crossing points may be included if the point of crossing is not controlled by a STOP or YIELD sign on the route in question, or does not have traffic signals.

4. PARKING: The prevailing speed may be reduced by 5 percent where parking is permitted adjacent to the traffic lanes.

5. MISCELLANEOUS: Other factors may be included in the investigation based on engineering judgment. Normally, isolated curves and turns, areas of restricted sight distances, no-passing zones, etc., should not to be considered as the basis for alteration of speed limits.

Selection of Altered Speed Limit

To determine the proposed altered speed limit, either use the calculated prevailing speed, or apply the percentage corrections resulting from any or all of the above optional factors to the prevailing speed, and select the closest 5 mile per hour increment. In no case, however, should the proposed altered limit differ either upward or downward from the prevailing speed by more than 9 miles per hour or by more than 20 percent, whichever is less. Next, compare the proposed altered speed limit to the speeds collected in the spot speed study and determine the anticipated violation rate. If the anticipated violation rate exceeds 50 percent, the proposed altered speed limit should be revised in 5 mile per hour increments until the anticipated violation rate is equal or less than 50 percent. If this results in a proposed altered speed limit which exceeds a 30 mph statutory speed for the highway in question, either the statutory speed or the proposed altered speed may be used to set the speed limits. If the speed selected results in a violation rate greater than 50 percent, the appropriate police agency(ies) should be notified that extra enforcement efforts may be necessary.

Differences in posted speeds between adjacent altered speed zones should not be more than 10 miles per hour.

C. Posting of General Speed Limits

Speed Reduction Signs

A Speed Reduction sign (W3-5) shall be erected in advance of any speed zone that is 10 miles per hour or more under the passenger car limit in a preceding statutory or altered limit of 45 miles per hour or more and should be erected at other locations where engineering judgment indicates the need. It shall be placed approximately 500 to 600 feet in advance of the lower speed zone and shall always be followed by a basic speed limit sign erected at the beginning of the zone.

On divided and one-way facilities having two or more lanes in one direction, the Speed Reduction signs, where used, and the first basic speed limit sign for the altered speed zone, shall be installed on both sides of the roadway except in situations where insufficient room exists in a median. Red 18-inch metal retroreflectorized "flags" shall be installed on the Speed Reduction signs preceding any transition from a 60 or 65 miles per hour zone to a lower speed zone.

When speed zones on rural highways extend only through signalized intersections, speed limit signs for the altered zones shall be installed at least 1,000 feet prior to the intersections on both sides of the roadway except in situations where insufficient room exists in a median. Normally, such altered zones should be terminated approximately 500 feet beyond the intersection.

Speed Limit Signs

Speed limit signs shall be posted at points of entry to the state even where the preceding speed limit in the adjacent state is the same. The signs should be placed as close to the state line as possible. On conventional rural highways, speed limit signs should also be posted after major highway intersections, and at such other locations as necessary to ensure that there is at least one sign every 10 miles. On Interstate highways and other full freeways, speed limit signs should be placed following the entrance ramps from all except very closely spaced interchanges, and at such other locations as necessary to ensure that there is at least one sign every 10 miles.

The prohibition on the use of electronic speed detection devices within 500 feet beyond certain speed limit signs in the direction of travel (Section 11-602 of the IVC) shall not be taken into account in the placement of speed limit signs.

The following spacings for speed limit signs are recommended in altered speed zones and for 30 mph zones in urban areas. All speed zones, either altered or statutory, shall be posted on state highways.

Posted Speed	Recommended Sign Spacing
30 mph or less	660 ft to 1,320 ft (2 to 4 blocks)
35 or 40 mph	990 ft to 1,980 ft (3 to 6 blocks)
45 mph	1,320 ft to 2,640 ft (4 to 8 blocks)
55 or 60	2 to 10 miles

Some speed limit signs for freeways/expressways where the speed limit differs between trucks over 4 tons and all other vehicles shall include an additional ‘Trucks Over 4 Tons’ R2-I109 plaque. This plaque shall be installed above the first 55 mph speed limit sign entering the dual speed zone and the first speed limit sign exiting the dual speed zone. Red 18-inch metal retroreflectorized flags shall also be installed on the first 55 mph speed limit sign entering a dual speed zone.

Minimum Speed Limit Signs

A MINIMUM 45 mph speed plaque (R2-I101) shall be placed below each basic 60 or 65 mph speed limit sign (R2-1) for fully access-controlled freeways only. It may be omitted where closely spaced interchanges or volume/capacity restraints make compliance with a 45 mph minimum speed limit impractical. A minimum speed shall not be used with 55 mph or lower speed limits.

SCHOOL SPEED LIMITS

School speed limits on highways under the jurisdiction of the department shall be established on the basis of Article VI of the Illinois Vehicle Code (IVC), Part 7 of the Illinois Manual on Uniform Traffic Control Devices (IMUTCD) and this policy.

Section 11-605 of the IVC allows establishment of 20 miles-per-hour speed limits on streets and highways passing schools or upon any street or highway where children pass going to and from school. Such established limit is to be in effect “On a school day when school children are present and so close thereto that a potential hazard exists because of the close proximity of the motorized traffic...” It further defines school days as beginning at 7 a.m. and ending at 4 p.m. Such a zone may be established for public, private and religious nursery, primary or secondary schools.

An engineering and traffic investigation shall be conducted to determine whether or not a school speed zone is warranted. The investigation shall consider such factors as the existing traffic control, whether school crosswalks are present or not, the type, character and volume of vehicular traffic, and the ages and numbers of schoolchildren likely to be present. It shall also consider where the children would be located in relation to the traffic.

Speed zones should be limited to those locations where school buildings or grounds devoted primarily to normal school day activities are adjacent to the highway or where groups of children cross the highway on their way to and from a school. Areas devoted primarily to athletic or other extracurricular activities should not be zoned.

The limits of school speed zones should be determined based upon where children are likely to be present and not based upon the limits of the school property. There are situations, primarily in rural areas, where the school-owned property line is some distance from the actual portion of the property occupied by the school and there are no children walking or present along that portion of the property. Establishing a 20 mile-per-hour school speed limit based solely on the location of the property line would be inappropriate. Conversely, it might be appropriate to impose a 20 mile-per-hour school speed limit some distance ahead of the property line where children walk close to the highway on their way to and from school and such path is part of a planned school walk route.

Speed zones should not be established for crossings where schoolchildren are protected by devices such as stop signs or traffic signals. An exception may be made when the speed zone serves to protect children walking on or immediately adjacent to the roadway in the school area.

Speed zones should not be established when the school or school grounds are completely isolated from the highway by means of a fence or other barrier, and no access to the highway is provided. They should also not be established for crossing where an underpass or overpass is provided or for school entrances used for buses or private vehicles carrying children to and from school.

The beginning of a school speed zone should be marked with a school speed limit 20 mph sign (S4-I100 or S4-I101) with a FINES HIGHER sign (R2-6P) mounted underneath. The end of a school speed zone should be marked with the appropriate standard speed limit sign (R2-1) and an END SCHOOL ZONE sign (S5-2) mounted underneath.

If requested by a local agency, CELL PHONE USE PROHIBITED signs (R2-I110) may be placed below Reduced School Speed Limit Ahead signs (S4-5) on state highways provided the local agency has a policy of placing such signs in conjunction with any school speed zones on roads under their jurisdiction. Where Reduced School Speed Limit Ahead signs are not used, the CELL PHONE USE PROHIBITED sign may be installed separately or below the school sign. (S1-1).

WORK ZONE SPEED LIMITS

A. Altered Speed Limits

- No Speed Limit Reduction or Work Zone Speed Limit– All roadway types

  The existing speed limit shall not be lowered and a work zone speed limit shall not be established when there is no lane reduction or apparent hazard.

- Existing 65 or 60 mph - Multilane:
  Speed Limit Reduction to 55 mph

  55 mph Work Zone Speed Limit signs (see Art. 701.14(b) of the Standard Specifications for Road and Bridge Construction) shall be used to reduce posted speed limits from 65 or 60 mph to 55 mph in construction work zones with lane closures or crossovers as shown on the Highway Standards or as noted in the traffic control plans. For this requirement to be added to an ongoing contract, it must be approved by the District Operations Engineer. Work Zone Speed Limit signs may also be used to reduce the existing speed limit to 55 mph if engineering judgment indicates the reduced speeds are necessary (See Section C). Approval of the District Operations Engineer is required.

- Existing 65 or 60 mph - Multilane:
  Speed Limit Reduction to 45 mph When Workers are Present

  45 mph Work Zone Speed Limit signs (see Art. 701.14(b) of the Standard Specifications for Road and Bridge Construction) within the lane closure shall be used when workers are present in the closed lane adjacent to traffic and are not protected by temporary concrete barrier. This sign may be used in conjunction with other Work Zone Speed signs to drop the 55 mph Work Zone Speed Limit to 45 mph. If conditions that warrant these signs develop during construction, the signs may be added to the contract upon approval of the District Operations Engineer (See Section C). These signs shall be utilized as indicated in the Highway Standards and as noted by the designer in the traffic control plans. The signs shall be covered, turned or removed when workers are no longer present.

- Existing 45 - 55 mph – Multilane:
  Work Zone Speed Limit 45 established

  Work Zone Speed Limit signs for existing multilane 45 to 55 mph speed limits shall be as shown on the Highway Standards and as noted in the traffic control plans. The signing changes an existing 45 mph speed limit to a 45 mph work zone speed limit. A reduction in the speed limit beyond 10 mph is not recommended and design changes should be considered that will allow traffic to safely move at 45 mph.

- Existing speed limit below 45 mph for multilane and all 2-Lane roadways

  The existing speed limit should not be lowered and a work zone speed limit should not be established.

  If a justification from Section C is met and cannot be corrected in a reasonable length of time, a 10 mph reduction may be considered. This reduction shall be based on engineering judgment and shall be approved by the District Operations Engineer.

B. Increased Fines in Work Zones

The applicable highway construction or maintenance speed limit fines are specified in Section 11-605.1 of the IVC.

The work zone must be posted according to the requirements for Work Zone Speed Limit signs. For the increased fines to be enforceable, the Minimum Fine Sign and the WORK ZONE Sign must be present as shown in the applicable Highway Standards.

C. Justifications for Work Zone Speed Limit Reductions

The following may be additional reasons for reducing an existing speed limit in a work zone or for establishing a work zone speed limit in excess of 10 mph below the existing speed limit. This reduction should be based on engineering judgment, documented, and approved by the District Operations Engineer.

- Narrow pavement lane width
- High traffic volumes
- Drop-offs
- Temporary road alignment where a design for higher speed operation is not feasible due to space requirements or other factors
- Inadequate sight distance

D. **Posting of Work Zone Speed Limit Signs**

Work Zone Speed Limit Signs shall be posted according to Article 701.14(b) of the Standard Specifications for Road and Bridge Construction, the applicable Highway Standards, and as shown on the design plans.

MISCELLANEOUS SPEED POLICIES

A. Blanket Speed Limit Signs

Posting of signs indicating general municipal speed limits, such as "SPEED LIMIT 25 ON VILLAGE STREETS," shall not be used on state highways. Section 11-604 of the IVC requires that speed limit signs be placed "...at the proper place or along the proper part or zone of the highway or street." The Office of Chief Counsel has determined that this requires each individual altered speed zone be signed.

B. Radar Warning Signs

SPEED RADAR TIMED, or other similar signs, shall not be used on state highways. An Illinois Attorney General's Opinion (1966-196) stated that such signs were not necessary for enforcement.

C. Aerial Speed Check Markings

Where requested by the Illinois State Police, aerial speed check markings on state highways may be placed in accordance with the guidelines contained in Section 7-401.21 of the Bureau of Operations Traffic Policies and Procedures Manual.

D. Design, Posted, and Operating Speeds

To prevent potential safety issues, the design speed selected to determine the design features of a roadway should equal or exceed the anticipated posted speed after construction as determined by the requirements of this policy. The designer should coordinate the design speed selection with the District Bureau of Operations anticipated posted speed limit selection. If the proposed design speed will be less than the anticipated posted speed, the designer must choose one of the following approaches:

- Seek a design exception
- Increase the design speed to equal the anticipated posted speed
- Post the project with a legal speed limit equal to the design speed
  (The legal speed limit shall be determined in accordance with:
  Section 625 ILCS 5/11-602 of the Illinois Vehicle Code
  Section 23 CFR 655 of the US Code of Federal Regulations
  The requirements of this policy)

The designer should avoid artificially selecting a design speed low enough to eliminate any design exceptions. For example, if IDOT criteria yield a design speed of 60 mph and one or more geometric features are adequate only for 55 mph, the design speed should be 60 mph and not 55 mph. The designer will then be required to seek design exceptions for 55 mph geometric features.

Curbed Sections

Sections with continuous barrier curbs at or near the edge of pavement should be avoided in areas where operating speeds can be expected to be greater than 45 mph. However, where a speed study justifies a speed limit of 50 mph or greater, the posted limit may be reduced to 45 mph upon the written approval of the District Operations Engineer. If the curbed section is short, such as with channelizing in conjunction with a freeway interchange, the operating speed should be used.

E. Two-Way Left Turn Lanes

Two-way left turn lanes should be avoided in areas where operating speeds can be expected to be greater than 45 mph. However, where a speed study justifies a speed limit of 50 mph or greater, the posted limit may be reduced to 45 mph upon the written approval of the District Operations Engineer.

F. Park Zone Speed Limits

Park Zone speed limits on roads under the jurisdiction of local agencies may be established on the basis of Section 11-605.3 of the IVC and part 2 of the Illinois Manual on Uniform Traffic Control Devices (IMUTCD).

Section 11-605.3 of the IVC allows local agencies to establish Park Zones and Park Zone Speed Limits by ordinance or resolution on streets and highways under their jurisdictions which abut parks. It does not allow the posting of a 20 mph Park Zone Speed Limit along streets or roads under the jurisdiction of the Illinois Department of Transportation.

A reduction in the speed limit along an abutting street under the jurisdiction of the department could be established in accordance with Section 11-602 of the IVC where warranted by a speed study. However, such a reduction in the speed limit would be signed as a normal speed limit and not as a "park zone speed."

If requested by local agencies, districts may post Illinois Standard W15-I100 PARK ZONE signs on abutting streets and highways under the jurisdiction of the department if the local agency has established and signed a park zone. These signs may be installed regardless of whether a "park zone speed limit" has been established or not.

SPOT SPEED STUDY

DIST: ______ CITY/LOCATION: ______ ROUTE: ______ DATE: ______ DAY: ______

CHECK NO.	RECORDER	HOURS FM: ___M TO: ___M	WEATHER	SURFACE WET DAMP DRY	___ FT. MI. E W N S OF	METER ON E W N S SIDE	TRAFFIC CHECKED: EB WB NB SB	85TH PERCNTLE	UPPER LIMIT 10 MPH PACE	POSTED LIMIT MPH	VIOLATION RATE

NUMBER OF VEHICLES

MPH	5	10	15	20	25	30	35	40	45
20									
21									
22									
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ABOVE 85 MPH, LIST INDIVIDUALLY: ______________________

ESTABLISHMENT OF SPEED ZONE
DISTRICT ________

ROUTE: ______________________ FROM: ____

TO: ______________________________________ LENGTH: __________________________

CITY: ______________________________ COUNTY: ________________________________

I SPOT SPEED STUDIES (Attached)

CHECK NO.	85^{TH} %	UPPER LIMIT 10 MPH PACE

V ACCESS CONFLICTS

RESIDENTIAL DRIVES: ______ X 1 = _____
SMALL BUSINESS DRIVES: ______ X 5 = _____
LARGE BUSINESS DRIVES: ______ X10 = _____
ACCESS CONFLICT NO. TOTAL: __________

STUDY LENGTH: ______ = ______________
(MILES) CONFLICTS / MILE

II TEST RUNS

RUN NO.	AVG. SPEED	DIRECTION
1		
2		
3		
4		
5		

VI MISC. FACTORS

PEDESTRIAN VOLUME: ________________
HIGH-CRASH LOCATION: _____ YES _____ NO
PARKING PERMITTED: _____ YES _____ NO

III PREVAILING SPEED

85^{TH} % AVG. : __________ MPH
UPPER LIMIT OF
10 MPH PACE: __________ MPH
TEST RUN AVE. : __________ MPH
PREVAILING SPEED: __________ MPH

VII PREVAILING SPEED ADJUSTMENT

DRIVEWAY ADJUSTMENT: ____________ %
PEDESTRIAN ADJUSTMENT: ____________ %
CRASH ADJUSTMENT: ____________ %
TOTAL (MAX 20%): ____________ %

________ MPH X _______ % = ____________
(Prevailing Speed) (adjust.) (Max. 9 MPH)

ADJUSTED PREVAILING SPEED: ____________

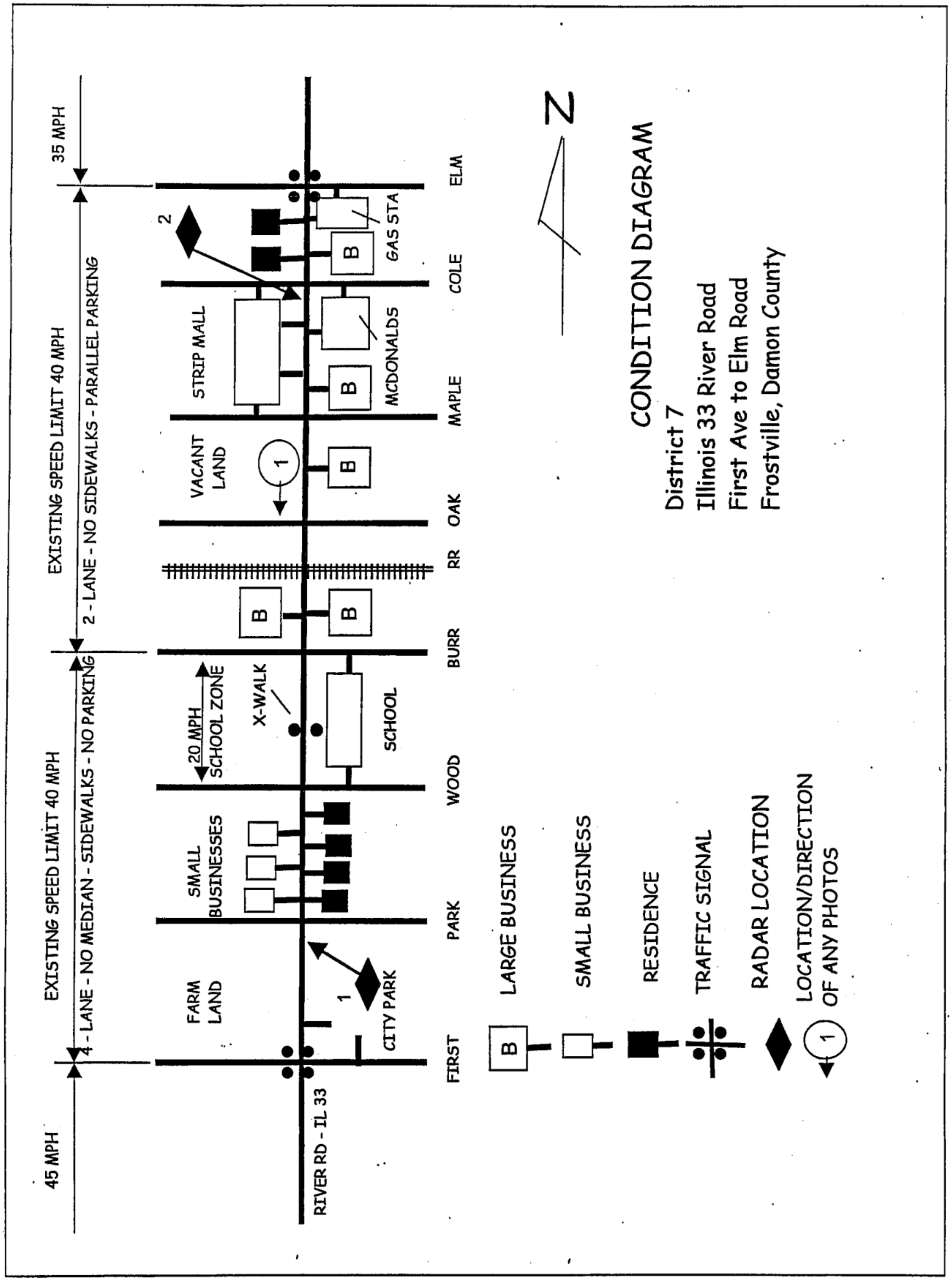
IV EXISTING SPEED LIMIT

ZONE BEING STUDIED: ________ MPH
VIOLATION RATE: ________ %
ADJACENT ZONE N or W: ______ MPH
LENGTH: ____________ MILES
ADJACENT ZONE S or E: ______ MPH
LENGTH: ____________ MILES

VIII REVISED SPEED LIMIT

RECOMMENDED SPEED LIMIT: ________ MPH
ANTICIPATED VIOLATION RATE: ________ %

RECOMMENDED BY: ____________________
DATE: ____________________
APPROVED BY: ____________________
DATE: ____________________



45 MPH
EXISTING SPEED LIMIT 40 MPH
4 - LANE - NO MEDIAN - SIDEWALKS - NO PARKING
EXISTING SPEED LIMIT 40 MPH
2 - LANE - NO SIDEWALKS - PARALLEL PARKING
35 MPH
RIVER RD - IL 33
FARM LAND
CITY PARK
1
SMALL BUSINESSES
20 MPH SCHOOL ZONE
X-WALK
SCHOOL
B
B
VACANT LAND
1
B
STRIP MALL
B
MCDONALDS
2
B
GAS STA
FIRST
PARK
WOOD
BURR
RR
OAK
MAPLE
COLE
ELM
N
CONDITION DIAGRAM
District 7
Illinois 33 River Road
First Ave to Elm Road
Frostville, Damon County
B
LARGE BUSINESS
SMALL BUSINESS
RESIDENCE
TRAFFIC SIGNAL
RADAR LOCATION
1
LOCATION/DIRECTION OF ANY PHOTOS