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Modeling Operating Speed

Synthesis Report

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Modeling Operating Speed

Synthesis Report

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Foreword

This report is a synthesis of existing operating speed models developed in different regions of the world. The models are grouped according to roadway type. Limitations and deficiencies in existing operating speed models and suggestions for future work are also identified. Practitioner perspectives on the potential use of speed prediction models in road design practice are provided from both the perspective of the United States and the international community.

The document was prepared by members and friends of the Transportation Research Board's Operational Effects of Geometrics Committee (AHB65). Yasser Hassan, Mohamed Sarhan, and Richard Porter served as the document editors. Individual chapters were written by Michael Dimaiuta, Eric Donnell, Alfredo Garcia, Yasser Hassan, Scott Himes, Paolo Perco, Richard Porter, Basil Psarianos, Mohamed Sarhan, and Mark Taylor. Their contributions and professional affiliations are noted at the start of each chapter.

The authors provided several rounds of reviews and revisions of the chapters. Additional peer review was provided by members and friends of the Transportation Research Board's Subcommittee on Performance-Based Analysis, including Kay Fitzpatrick, Richard Knoblauch, Ray Krammes, Geoff Millen, and Peter Park. The report was approved for publication by these same authors, peer reviewers, and the committee chair in July 2010.

—Raymond A. Krammes, Immediate Past Chair
TRB Operational Effects of Geometrics Committee

Contents

CHAPTER 1

Introduction

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Speed can be described as one of the most important factors that road users consider to evaluate the convenience and efficiency of a certain route. In addition, along with other factors such as travel time and cost, speed may affect the decisions made by drivers in selecting between different route alternatives. Speed has also been recognized as one of the measures that designers can use to examine road consistency and driver expectancy on roadways.

In North American design guides, highway elements are designed by selecting a design speed that is consistent with the anticipated operating speed. In general, the design speed is selected with respect to road class, topography, and land use. Designers are generally encouraged to retain a constant design speed over a substantial length of roadway as a means to promote design consistency. The main purpose of this guidance is to reduce frequent changes in the road configurations and hence achieve a harmonious driving environment.

Applications of this design speed concept have proven in many instances in the literature to be insufficient to solely reduce speed variations. The reason is the existence of design elements, e.g., cross section dimensions, which are not directly related to or are inelastic with design speed but still have a significant impact on operating speeds selected by road users. Therefore, including operating speed prediction steps in the design phase of the project development process has been the focus of significant inquiries in recent years. The assessment of operating speeds affords the opportunity to assess the expected speed changes of individual vehicles traversing successive road elements. By reducing such speed changes, there is a greater chance of enhancing traffic flow and improving safety performance.

Several factors influence operating speeds. This document focuses on factors of interest to highway designers and highway and traffic engineers, i.e., “the physical characteristics of the highways, the amount of roadside interference, the weather, the presence of other vehicles, and the speed limitation” (AASHTO 2004). There is a large body of published literature that presents operating speed as a function of road parameters such as horizontal curve radius, vertical grade, rate of vertical curvature, traffic flow characteristics, and cross sectional dimensions. Most of the studies were carried out in North America and Europe focusing mainly on the influence of horizontal curvature on free-flow speeds selected by road users. The influence of the vertical alignment was also reported by some studies to have a significant impact on speeds, especially those of heavy vehicles. This report provides a comprehensive discussion of operating speed models based on a review of published literature conducted by members and friends of the TRB Operational Effects of Geometrics Committee.

The objectives of the report are to identify and document existing operating speed models developed in different regions of the world. The models are grouped according to roadway type. In addition, authors of the report identify several limitations and deficiencies in the existing operating speed models, and with that, offer suggestions for future work. Practitioner perspectives on the potential use of speed prediction models in road design practice are provided from both a North American and International perspective. The remaining chapters are organized as follows:

- Chapter 2 describes past studies that were conducted in North America;
- Chapter 3 presents the current speed modeling practice in several European countries;
- Chapter 4 covers numerous studies in regions of the world other than North America and Europe;
- Chapter 5 outlines limitations and deficiencies in existing speed models; and
- Chapter 6 provides a summary, conclusions, and recommendations for future speed modeling research and implementation.

CHAPTER 2

Speed Models in North America

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TWO-LANE RURAL HIGHWAYS

There are numerous studies in North America that presented models to predict 85th percentile, free-flow speed in terms of the road geometry. For example, Lamm et al. (1987, 1988, and 1990) developed a model to predict 85th percentile speed on horizontal curves. Data were collected from a set of curves with a wide range of characteristics such as intersection spacing and vertical grade. All road segments had paved shoulders and average annual daily traffic (AADT) volumes ranging from 400 to 5,000 vehicles per hour (vph). In addition, data were collected for different vehicle types including passenger cars, pickups, vans, and trucks. Geometric data were also collected at the selected sites. Predictor variables considered in the operating speed model included degree of horizontal curve, lane width, length of horizontal curve, shoulder width, superelevation, available sight distance, vertical grade, posted speed limit, and AADT. Ordinary least squares (OLS) regression with stepwise specification procedure was used to model the 85th percentile speed as follows:

$$V85 = 34.70 - 1.00(\mathrm{DC}) + 2.081(\mathrm{LW}) + 0.174(\mathrm{SW}) + 0.0004(\mathrm{AADT})$$

where

- $V85$ = expected 85th percentile speed on horizontal curves (mph)
- DC = degree of curve (range 0° to 27° per 100 ft of arc)
- LW = lane width (ft)
- SW = shoulder width (ft)
- AADT = average annual daily traffic (vehicles per day)

The model is significant (i.e., the model provides more explanation than a model with only a constant) at a 95% confidence level with a coefficient of determination (R^2) of 0.842. The statistical analysis showed that factors such as the superelevation and posted speed limit were highly correlated with the degree of curve. Therefore, those two variables were excluded from the model. Lane width, shoulder width, and AADT were all statistically significant, but only explained about 5.5% of the variation in operating speeds. Other models were also estimated by Lamm et al. (1988). The models were categorized with respect to lane widths as follows in Table 1.

Morrall and Talarico (1994) also estimated a model that describes $V85$ of passenger cars on horizontal curves in terms of the degree of curve. The model and coefficient of determination were as follows:

$$V85 = e^{(4.561-0.00586\mathrm{DC})} \qquad R^2 = 0.631$$

where

$V85$ = expected 85th percentile operating speed (km/h)
DC = degree of curve (degrees/100 m of arc)

Operating speed on horizontal curves was also studied by Islam and Seneviratne (1994). The authors collected data on eight horizontal curves in the Logan Canyon section of Highway 89 in Northeastern Utah. The road is a two-lane rural highway with degrees of curvature ranging from 4 to 28 degrees. For each curve, 125 spot speeds were measured at three points including the point of curvature (PC), midpoint of the curve (MC), and the point of tangency (PT). The estimated models were as follows:

$$\mathrm{V85_{PC}} = 95.41 - 1.48\mathrm{DC} - 0.012\mathrm{DC}^2 \qquad R^2 = 0.99$$
$$\mathrm{V85_{MC}} = 103.30 - 2.41\mathrm{DC} - 0.029\mathrm{DC}^2 \qquad R^2 = 0.98$$
$$\mathrm{V85_{PT}} = 96.11 - 1.07\mathrm{DC} \qquad R^2 = 0.98$$

where

$V85_{\mathrm{PC}}$ = expected 85th percentile speed at PC (km/h)
$V85_{\mathrm{MC}}$ = expected 85th percentile speed at MC (km/h)
$V85_{\mathrm{PT}}$ = expected 85th percentile speed at PT (km/h)
DC = degree of curve (degrees per 30 m of arc)

TABLE 1 Modeling *V*85 at Different Values of Lane Width (Lamm et al., 1988)

Lane width (ft)	*V*85	R^2
All	58.656 – 1.135 *DC*	0.787
	25.314 + 0.554 *RS*	0.719
10	55.646 – 1.019 *DC*	0.753
	27.173 + 0.459 *RS*	0.556
11	58.310 – 1.052 *DC*	0.746
	29.190 + 0.479 *RS*	0.744
12	59.746 – 0.998 *DC*	0.824
	26.544 + 0.562 *RS*	0.835

NOTE: Where RS = posted speed on curve (mph); DC = degree of horizontal curve

Islam and Seneviratne (1994) concluded that the radius of curve is the most significant parameter in predicting operating speed on horizontal curves. It was also noted that, for the same horizontal curve, there were significant differences between the operating speeds at the point of curvature (PC), point of tangency (PT), and the midpoint of the curve (MC) (Gibreel et al., 1999). These differences were found to increase as the degree of curvature increased. Therefore, Gibreel et al. (1999) suggested that speed consistency problems may tend to arise along sharp horizontal curves.

Krammes et al. (1995) collected speed data for passenger cars on a sample of horizontal curves and approach tangents. All curves were on rural two-lane highways located in three geographic regions of the United States.

1. East: New York and Pennsylvania.
2. West: Washington and Oregon.
3. South: Texas.

As shown in Table 2, observed speeds on long tangents were assumed to represent desired speeds. The models estimated and reported in the study are shown below:

$$V85 = 103.66 - 1.95\mathrm{DC} \qquad R^2 = 0.80$$
$$V85 = 102.45 - 1.57\mathrm{DC} + 0.0037L - 0.10I \qquad R^2 = 0.82$$
$$V85 = 41.62 - 1.29\mathrm{DC} + 0.0049L - 0.12I + 0.95V_t \qquad R^2 = 0.90$$

where

$V85$ = expected 85th percentile speed on horizontal curves (km/h)
DC = degree of curve (degrees per 30 m of arc)
L = length of curve (m)
I = deflection angle (degrees)
V_t = measured 85th percentile speed on approach tangent (km/h)

TABLE 2 Mean 85th Percentile Speeds (km/h) on Long Tangents by Region and Terrain (Krammes et al., 1995)

Region	**Terrain**		
	Level	Rolling	All Terrain
South	102.4*	99.2	99.8
East and West	97.9	95.5*	96.0
All Regions	99.8	96.6	97.9

* Difference is significant at $\alpha = 0.05$

The data collected by Krammes et al. (1995) were further evaluated by McFadden et al. (2001). Two back propagation artificial neural network (ANN) models were used. Data from 100 sites (approximately two thirds of the data) were used for network training. The remaining 38 sites were used for model validation. The structure of the two models and their explanatory power are summarized in the Table 3. The models were also compared to OLS regression models estimated by Krammes et al. (1995) using the same data. It was concluded that "these comparisons found that ANNs offer predictive powers comparable with those of regression and that ANNs are able to overcome many of the assumptions and limitations inherent to linear regression." The comparisons are also shown in Table 3.

Furthermore, Voigt and Krammes (1996) developed another set of models that predict *V*85 of passenger cars at the midpoint of horizontal curves. The speed and geometric data used for modeling included a total of 138 simple circular curves and 78 approach tangents on rural two-lane highways in five states including New York, Pennsylvania, Oregon, Washington, and Texas. For each site, *V*85 was estimated based on a minimum of 100 free-flow passenger vehicle speeds. The explanatory variables included the degree of curve, length of curve, superelevation rate, and deflection angle.

$$V85 = 102.0 - 2.08\text{DC} + 40.33e \qquad R^2 = 0.81$$
$$V85 = 99.6 - 1.69\text{DC} + 0.014L - 0.13 - \text{Delta} + 71.82e \qquad R^2 = 0.84$$

where

$V85$ = 85th percentile speed at midpoint of curve (km/h),
e = superelevation rate (m/m),
L = length of curve (m), and
Delta = deflection angle (degrees).

TABLE 3 Comparison of ANN and OLS Models (McFadden et al., 2001)

	ANN Model 1	ANN Model 2	Krammes et al. (1995) Model 1[a]	Krammes et al. (1995) Model 2[b]
Training data	$R^2 = 0.68$	$R^2 = 0.73$	$R^2 = 0.80$	$R^2 = 0.82$
Validation data	$R^2 = 0.76$	$R^2 = 0.79$		
Input variables	Degree of curve	Degree of curve Length of curve Deflection angle	Degree of curve	Degree of curve Length of curve Deflection angle
Output variable	Expected 85th percentile speed	Expected 85th percentile speed	Expected 85th percentile speed	Expected 85th percentile speed

NOTES:
[a] *V*85 = 103.66 – 1.95 DC; where *V*85 = expected 85th percentile speed on horizontal curves (km/h); DC = degree of curve (degrees per 30 m of arc)
[b] V85 = 102.45 – 1.57 DC + 0.0037*L* – 0.10 Delta, where *L* = length of curve (m), Delta = deflection angle (degrees), *V*85, and DC as previously defined

The authors concluded that the superelevation rate is statistically significant, but it explains only 1% to 2% of additional variability in the data than regression models including only radius or degree of curvature.

Passetti and Fambro (1999) investigated the effect of spiral transitions on curve speeds by collecting data from 51 sites (12 spiral transition curves and 39 circular curves) across six states (New York, Pennsylvania, Oregon, Washington, Minnesota, and Texas). More than 100 spot speeds were collected at each site. Stepwise OLS regression technique was used to estimate the following model:

$$V85 = 103.9 - 3020.5\left(\frac{1}{R}\right) \qquad R^2 = 0.68$$

where

$V85$ = 85th percentile speed for curves with or without spiral transitions (km/h), and
$\left(\frac{1}{R}\right)$ = inverse of curve radius (1/m).

Passetti and Fambro (1999) concluded that, for the range of data analyzed, spiral transitions did not significantly influence the speed at which passenger cars traversed a horizontal curve on rural two-lane highways. However, spiral transition curves may tend to affect vehicle speeds as the curve radius decreases. In addition, based on the literature reviewed and the research conducted in this study, the inclusion of spiral transitions in horizontal curve design did not produce significant operational benefits for passenger cars. A correlation analysis showed that the inverse of curve radius, curve length, superelevation rate, and deflection angle had high correlation coefficients and should not be used in the same regression equation.

Another study was done by McFadden and Elefteriadou (2000) for the purpose of using $V85$ profiles to evaluate design consistency on two-lane, rural highways. Speed data were collected at 21 horizontal curves (12 curves in Pennsylvania and nine curves in Texas). The 85th-percentile maximum speed reduction (85MSR) was modeled as a function of road geometry. The following model, applicable to passenger cars, was estimated with OLS regression:

$$85\text{MSR} = -14.90 + (0.144\, V85_{@\text{PC}200}) + (0.0153\ \text{LAPT}) + \left(\frac{954.55}{R}\right) \qquad R^2 = 0.71$$

where

85MSR = 85th percentile speed reduction on curve (km/h);
$V85_{@\text{PC}200}$ = 85th percentile speed at 200 m prior to point of curvature (km/h);
R = horizontal curve radius (m); and
LAPT = length of approach tangent (m).

Another model was estimated and recommended for cases where the approach tangent speed is unknown:

$$85\text{MSR} = -0.812 + \left(\frac{998.19}{R}\right) + (0.017 * \text{LAPT}) \qquad R^2 = 0.60$$

The 85MSR was calculated by evaluating each driver's speed profile while the vehicle traverses from an approach tangent through a horizontal curve. The maximum speed reduction was then determined for each vehicle. The parameter 85MSR was also compared with the difference in 85th percentile speeds (85S). It was stated that 85MSR is significantly larger than 85S. The data showed that, on average, 85MSR is approximately twice as large as 85S (McFadden and Elefteriadou, 2000). In addition, the correlation analyses for approach tangent variables showed that a correlation exists between speed reduction into the curve, length of approach tangent, pavement width, shoulder width, and posted speed limit (McFadden and Elefteriadou, 2000). Finally, the study emphasized the conclusion reported by Hirshe (1987), that the use of 85th percentile speeds by operating-speed profile models to evaluate design consistency underestimates the amount of speed reduction experienced by individual drivers (McFadden and Elefteriadou , 2000).

A study by Fitzpatrick et al. (2000a) also resulted in models of *V*85 for passenger cars operating on different combinations of horizontal and vertical alignments. Free-flow speed data were collected at 176 sites in six states including Minnesota, New York, Pennsylvania, Oregon, Washington, and Texas. Models were estimated for 7 out of 10 identified alignment conditions.

Table 4 shows that, in most cases, *V*85 was predicted using the inverse of the horizontal curve radius or the inverse of the rate of vertical curvature. In cases where the sample size was too small to estimate a model, the desired speed was assumed to be 100 km/h, based on the earlier study by Krammes et al. (1995). Findings from the study can be summarized as follows (Fitzpatrick et al., 2000a).

Speeds on Curves: Horizontal Curves on Grades

- For passenger vehicles, curve radius was the only statistically significant independent variable in predicting *V*85 for all alignment combinations that included a horizontal curve on a grade. The recommended functional form of the independent variable in the regression models was $1/R$.
- Operating speeds on horizontal curves are very similar to speeds on long tangents when the radius is greater than or equal to approximately 800 m. Under this condition, the grade of the section may control the selection of speeds, and the effect of the horizontal radius on *V*85 is negligible.
- Operating speeds on horizontal curves drop sharply when the radius is less than 250 m.

Speeds on Curves: Vertical Curves on Horizontal Tangents

- Passenger car speeds on vertical curves combined with horizontal tangents with limited sight distance could be predicted using the rate of vertical curvature as the independent variable. The recommended functional form of the independent variable in the regression models was $1/K$.
- A statistically significant regression model could not be found for crest curves where the sight distance is not limited. Therefore, the desired speed for long tangents is assumed.
- For sag curves on horizontal tangents, regression analysis indicated that the desired speed on long tangents should be assumed.

TABLE 4 *V*85 Prediction Models on Two-Lane Rural Roads (Fitzpatrick et al., 2000b)

Equation No.	Alignment Condition	Formula	No. of Sites	R^2	MSE
1	Horizontal curve on grade: $-9\% \le g \le -4\%$	$V85 = 102.10 - \frac{3077.13}{R}$	21	0.58	51.95
2	Horizontal curve on grade: $-4\% \le g \le 0\%$	$V85 = 105.98 - \frac{3709.90}{R}$	25	0.76	28.46
3	Horizontal curve on grade: $0\% \le g \le 4\%$	$V85 = 104.82 - \frac{3574.51}{R}$	25	0.76	24.34
4	Horizontal curve on grade: $4\% \le g \le 9\%$	$V85 = 96.61 - \frac{2752.19}{R}$	23	0.53	52.54
5	Horizontal curve combined with sag vertical curve	$V85 = 105.32 - \frac{3438.19}{R}$	25	0.92	10.47
6	Horizontal curve combined with nonlimited sight distance crest vertical curve ($k > 43$ m/%)[a]	[a]	13	n/a	n/a
7	Horizontal curve combined with limited sight distance crest vertical curve ($k \le 43$ m/%)[b]	$V85 = 103.24 - \frac{3576.51}{R}$	22	0.74	20.06
8	Sag vertical curve on horizontal tangent	*V*85 = assume desired speed	7	n/a	n/a
9	Vertical crest curve with nonlimited sight distance on horizontal tangent	*V*85 = assume desired speed	6	n/a	n/a
10	Vertical crest curve with limited sight distance on horizontal tangent	$V85 = 105.08 - \frac{149.69}{K}$	9	0.60	31.10

NOTES:

AC EQ# = alignment condition equation number;
*V*85 = expected 85th percentile speed of passenger cars at curve midpoint (km/h);
R = radius of horizontal curve (m);
K = rate of vertical curvature (K); and
G = grade (%).

a. Use lowest of the speeds predicted from Equations 1 or 2 (for the downgrade) and Equations 3 or 4 (for the upgrade).

b. Check the speeds predicted from Equations 1 or 2 (for the downgrade) and Equations 3 or 4 (for the upgrade) and use the lowest speed. This will ensure that the speed predicted along the combined curve will not be better than if just the horizontal curve was present (i.e., that the inclusion of a limited sight distance crest vertical does not result in a higher speed).

Speeds on Curves: Horizontal Curves Combined with Vertical Curves

- For non-limited sight-distance crest vertical curves combined with horizontal curves, the expected *V*85 speed is the lowest of those predicted using the models estimated for horizontal curves on grades or the assumed desired speed. The collected speed data for this condition were generally on horizontal curves with large radii and most speeds were above 100 km/h.
- For horizontal curves combined with either sag or limited sight-distance crest vertical curves, the radius of the horizontal curve was the best predictor of speed.

Speeds on Curves: Spirals

- The data showed that the existence of spiral transitions had no significant effect on *V*85 at the midpoint of horizontal curves.

Speeds on Curves: Other Vehicle Types

- The data for all truck types and recreational vehicles on horizontal curves display a general speed behavior that is similar to that of passenger vehicles.
- The influence of grade on trucks and recreational vehicles is similar to, but larger than, the effects observed for passenger vehicles.

Speeds on Tangents

As for speeds on tangents, none of the alignment indices included in the study was statistically significant in modeling desired speeds on long tangents of two-lane rural highways. In addition, there were significant regional differences in the desired speeds of motorists on long tangents of two-lane rural highways. Moreover, out of all geometric variables examined, only the vertical grade at the tangent site significantly affected the desired speeds of motorists on long tangents of two-lane rural highways. Furthermore, the average *V*85 per state for long tangents ranged from 93 to 104 km/h. Based on the data and engineering judgment, the operating speeds on tangents and the maximum operating speeds on horizontal curves could be rounded to 100 km/h.

Two-Lane with Passing Vehicle-Performance Equations

- The two-lane with passing (TWOPAS) vehicle-performance equations can be used to determine the speed for specific vehicle types at any point on a grade.

Acceleration and Deceleration Rates for a Horizontal Curve

- The validation results indicate that the acceleration and deceleration assumptions employed in the speed-profile model presented in FHWA-RD-94-034 (0.85 m/s^2 for both acceleration and deceleration) are not valid for the set of study sites selected in this study. The speed values predicted and the speed values measured in the field were statistically different at the 95 % confidence level.
- The only sites with acceleration and deceleration rates that approached 0.85 m/s^2 were those with curve radii less than 250 m.

- New models were estimated to consider the effect of curve radii on the acceleration/deceleration rates. The models were based on maximum acceleration and deceleration rates observed at the study sites.
- For deceleration rates, a linear regression model was recommended for curves with $R >$ 175 m, with assumptions of -1.0 m/s^2 deceleration rate for curves with radii < 175 m and 0 m/s^2 for radii >436 m (see Table 5). In addition, the recommended acceleration model is a step-function rather than a regression-type model. Regression analyses resulted in very low R^2 for all analysis alternatives, and thus constant (average) rates for sites based on their curve radii was recommended as shown in Table 5.

Validation of the Speed-Prediction Equations

- The validation of the six speed-prediction equations for horizontal and vertical curves (see Table 4) was performed by comparing the formulas predicting $V85$ to field observations at the midpoint of horizontal curves. The overall mean absolute percent error for the six equations was 5.7%.
- Validation was also conducted for the predicted speed change between the midpoint of the approach tangent and the midpoint of the horizontal curve. In general, the models were found to differ from the observed change in speed between the tangent and the horizontal curve by an average of 98%.

Speed-Profile Model

- A speed-profile model was developed that can be used to generate a speed profile along an alignment. The lowest anticipated speed would be obtained by selecting the lowest speed predicted by the selected desired speed on the approach tangent, the speed-prediction equations, and/or the TWOPAS model. This speed would then be adjusted prior to and departing from curves using the acceleration and deceleration values determined in the study.
- In another study done by Polus et al. (2000), the authors analyzed the variability of the operating speeds on 162 tangent sections of two-lane rural highways. In addition, operating speed models were estimated based on the geometric characteristics of the study sites. The results are as follows.

TABLE 5 Deceleration and Acceleration Rates at Different Curve Radius (Fitzpatrick et al., 2000a)

Radius of Curvature (m)	**Deceleration Rate (m/s^2)**	**R^2**	**MSE**
$R < 175$ m	–1.0	N/A	N/A
175 m $< R <$ 436 m	$0.6794 - 295.14/R$	0.4778	1.65
436 m $< R$	0.0	N/A	N/A
Radius of curvature (m)	Acceleration Rate (m/s^2)		
175 m $< R <$ 250 m	0.54		
250 m $< R <$ 436 m	0.43		
436 m $< R <$ 875 m	0.21		
875 m $< R$	0.0		

Geometric Measure Models

$$V85 = 102.5 - \frac{37.34}{e^{0.00668\mathrm{GM}_l}}; \qquad \mathrm{GM}_l \leq 200 \qquad R^2 = 0.33$$
$$V85 = 105.0 - \frac{21.30}{e^{0.00092\mathrm{GM}_l}} \qquad \mathrm{GM}_l \leq 1000 \qquad R^2 = 0.23$$
$$\mathrm{GM}_l = \left[\frac{\mathrm{TL}(R_1 \times R_2)^{1/2}}{100}\right] \qquad \mathrm{TL} \geq t$$

where

$V85$ = 85th percentile speed (km/h);
GM_l = geometric measure of tangent section and attached curves for long tangents (m^2);
TL = tangent length (m);
R_1, R_2 = previous and following curve radii (m); and
t = selected threshold for tangent length (m).

Group Models

Group I Conditions: small radii (R_1 and R_2 < 250 m) and small *TL* (*TL* ≤ 150 m).

$$V85 = 101.11 - \frac{3420}{\mathrm{GM}_s} \qquad R^2 = 0.55$$
$$\mathrm{GM}_s = \frac{R_1 + R_2}{2}$$

Group II Conditions: small radii (R_1 and R_2 < 250 m) and intermediate TL (TL = 150 to 1,000 m).

$$V85 = 98.405 - \frac{3184}{\mathrm{GM}_l} \qquad R^2 = 0.68$$
$$\mathrm{GM}_l = \left[\frac{\mathrm{TL}(R_1 \times R_2)^{1/2}}{100}\right]$$

If maximum 85th percentile speed is established as 105 km/h:

$$V85 = 105.00 - \frac{28.107}{e^{0.00108\mathrm{GM}_l}} \qquad R^2 = 0.74$$

Group III Conditions: intermediate radii (R_1 and R_2 > 250 m) and intermediate TL (TL = 150 to 1,000 m). No successful models were identified.

$$V85 = 97.73 + 0.00067\mathrm{GM} \qquad R^2 = 0.20$$

Group IV Conditions: large TL (TL > 1,000 m) and any reasonable radii.

$$V85 = 105.00 - \frac{22.953}{e^{0.00012\mathrm{GM}_l}} \qquad R^2 = 0.84$$

The highest reasonable speed on short tangents is influenced primarily by the controlling geometry of the preceding and succeeding curves. On long tangents, however, the speed is influenced by supplemental factors (e.g., speed limit and level of enforcement) and secondary geometric variables (e.g., cross section and vertical grade). The combination of all these variables would make the prediction of *V*85 on tangents a relatively complex task (Polus et al., 2000). In addition, the analyses showed that, when determining *V*85 at the middle of a tangent section, it is necessary to observe a longer section that includes the preceding and succeeding curves because these constitute the primary variables affecting speed. The influence of other secondary geometric variables was investigated and found not to affect speed as much as do the primary variables (Polus et al., 2000). Moreover, the models in Group I and Group II provided a good fit to the data and could be adapted for prediction purposes during the planning process for new two-lane highways. The models for Group III and Group IV were preliminary, and they clearly need additional data. Further research was recommended to determine the effect of cross-sectional elements (i.e., lane width and roadside characteristics) and the direction of preceding curves on tangent operating speeds.

Using a combination of field data and simulation-generated data, Donnell et al. (2001) developed models for *V*85 of trucks on horizontal curves of two-lane rural highways. The TWOPAS traffic simulation model was used to generate the simulation data. Thirteen models were reported as shown in Table 6. The models consider the effect of length and grade of approach tangents, horizontal curve radius, and length and grade of departure tangents.

From 200 to 100 m before the horizontal curve, the radius of curve, length of approach tangent, grade of approach tangent, and the length of approach tangent radius interaction term were all statistically significant truck speed predictors. Fifty meters before the horizontal curve, the interaction term was no longer statistically significant while the main effects remained. Beginning at the point of curvature and continuing through the point of tangency, the radius, grade of departure tangent, and length of departure tangent were the statistically significant variables in the truck speed prediction models. Beyond the horizontal curve, both the grade and length of the departure tangent were statistically significant. All of the speed prediction models had a coefficient of determination between 0.552 and 0.627. Based on the outcome of the study, the following was concluded by Donnell et al. (2001):

- TWOPAS adequately simulates operating speeds of passenger cars on segments containing horizontal curves using the criterion of ±7 km/h. In general, TWOPAS tends to overestimate the speed drop caused by the presence of the horizontal curve.
- In TWOPAS, radius contributed more to the speed drop for passenger cars, and grade had a bigger effect on simulated truck speeds. The speed profiles for passenger cars and trucks show similar trends, both in the field and in simulations, with the speeds of trucks somewhat lower than the passenger cars.
- A series of regression models was developed to predict 85th percentile speed along a horizontal curve with varying design characteristics. These regression models mostly consider

TABLE 6 Speed Models as Reported by Donnell et al. (2001)

Location	Model	R^2
PC 200:	$V85 = 51.5 + 0.137(R) – 0.779(GAPT) + 0.0127(LAPT) – 0.000119(LAPT * R)$	$R^2 = 0.622$
PC 150:	$V85 = 54.9 + 0.123(R) – 1.07(GAPT) + 0.0078(LAPT) – 0.000103(LAPT * R)$	$R^2 = 0.627$
PC 100:	$V85 = 56.1 + 0.117(R) – 1.15(GAPT) + 0.0060(LAPT) – 0.000097(LAPT * R)$	$R^2 = 0.613$
PC 50:	$V85 = 78.7 + 0.0347(R) – 1.30(GAPT) + 0.0226(LAPT)$	$R^2 = 0.552$
PC:	$V85 = 78.4 + 0.0140(R) – 1.40(GDEP) – 0.00724(LDEP)$	$R^2 = 0.562$
QP:	$V85 = 75.8 + 0.0176(R) – 1.41(GDEP) – 0.0086(LDEP)$	$R^2 = 0.600$
MP:	$V85 = 75.1 + 0.0176(R) – 1.48(GDEP) – 0.00836(LDEP)$	$R^2 = 0.600$
3QP:	$V85 = 74.7 + 0.0176(R) – 1.59(GDEP) – 0.00814 (LDEP)$	$R^2 = 0.611$
PT:	$V85 = 74.5 + 0.0176 (R) – 1.69(GDEP) – 0.00810(LDEP)$	$R^2 = 0.611$
PT 50:	$V85 = 82.8 – 2.00(GDEP) – 0.00925 (LDEP)$	$R^2 = 0.564$
PT 100:	$V85 = 83.1 – 2.08(GDEP) – 0.00934(LDEP)$	$R^2 = 0.577$
PT 150:	$V85 = 83.6 – 2.29(GDEP) – 0.00919(LDEP)$	$R^2 = 0.604$
PT 200:	$V85 = 84.1 – 2.34(GDEP) –0.00944(LDEP)$	$R^2 = 0.607$

where
$V85$ = expected 85th percentile speed (km/h),
R = radius of horizontal curve (m),
GAPT = grade of approach tangent (%),
LAPT = length of approach tangent (m),
GDEP = grade of departure tangent (%),
LDEP = length of departure tangent (m),
PC # = number of meters before the curve PC,
PT # = number of meters after the curve PT, and
QP, MP, 3QP = quarter point, midpoint and three-quarter point of curve, respectively.

the effect of horizontal curves on trucks, coupled by the effect of grades. They did not consider the effect of combinations of vertical and horizontal curvature, and thus their application is not recommended for use at such sites.

Gibreel et al. (2001) developed operating speed models for two-lane rural highways that account for the three-dimensional (3-D) nature of highways. Two types of 3-D combinations were considered: a horizontal curve combined with a sag vertical curve and a horizontal curve combined with a crest vertical curve. Regression analysis was used to estimate the operating speed models (see Table 7) based on data collected on Highway 61 and Highway 102 in Ontario, Canada. The results show that there is a significant difference between the predicted operating speed using the 2-D and 3-D models.

TABLE 7 Speed Models as Reported by Gibreel et al. (2001)

Location	Model	R^2
Sag Curves		
AT	$V85 = 91.81 + 0.010R + 0.468(\text{Sqrt. } L_V) - 0.006G_1^3 - 0.878 \ln(A) - 0.826 \ln(L_0)$	0.980
BC	$V85 = 47.96 + 7.216 \ln(R) + 1.534 \ln(L_V) - 0.258\, G_1 - 0.653\, A + 0.02\, e^E - 0.008\, L_0$	0.980
MC	$V85 = 76.42 + 0.023\, R + 0.00023\, K^2 - 0.008\, e^A + 0.062\, e^E - 0.00012\, L_0^2$	0.940
EC	$V85 = 82.78 + 0.011\, R + 2.068 \ln(K) - 0.361\, G_2 + 0.036\, e^E - 0.00011\, L_0^2$	0.950
DT	$V85 = 109.45 - 1.257\, G_2 - 1.586 \ln(L_0)$	0.790
Crest Curves		
AT	$V85 = 82.29 + 0.003\, R - 0.05\, \text{DFC} + 3.441 \ln(L_V) - 0.533\, G_1 + 0.017\, e^E - 0.000097\, L_0^2$	0.940
BC	$V85 = 33.69 + 0.002\, R + 10.418 \ln(L_V) - 0.544\, G_1 + [8.699/ \ln(1 + A)] + 0.032\, e^E - 0.011\, L_0$	0.970
MC	$V85 = 26.44 + 0.251(\text{Sqrt. } R) + 10.381 \ln(L_V) - 0.423\, G_1 + [6.462/ \ln (1 + A)] + 0.051\, e^E - 0.028\, L_0$	0.980
EC	$V85 = 74.97 + 0.292\, (\text{Sqrt. } R) + 3.105 \ln(K) - 0.85\, G_2 + 0.026\, e^E - 0.00017\, L_0^2$	0.900
DT	$V85 = 105.32 - 0.418\, G_2 - 0.123\, (\text{Sqrt. } L_0)$	0.830

where

AT = approach tangent,
DT = departure tangent,
BC = beginning of curve,
MC = middle of curve,
EC = end of curve,
R = radius of curvature (m),
L_v = length of vertical curve (m),
L_0 = distance between horizontal and vertical points of intersection (m),
G_1, G_2 = first and second grade in direction of travel, respectively (%),
A = algebraic difference in grades (%),
e, E = superelevation rate (%),
K = length of vertical curve for 1% change in grade (m), and
DFC = deflection angle of curve (degrees).

Jessen et al. (2001) modeled $V50$, $V85$, and $V95$ free-flow speeds of passenger cars on crest vertical curves located on horizontal tangents. Data were collected at 70 sites on two-lane rural roads in Nebraska, with at least 275 observations per site. Speeds were observed at a control location (i.e., before the curve) and on the curve at the location of minimum available sight distance. Sixty-two of the sites were used for model estimation. The models reported by Jessen et al. (2001) are presented in Table 8. The authors stated that the approach grade affected vehicle speeds at the location with minimum available sight distance along the curve (called the limit location by the authors). As the approach grade increased, the expected $V50$, $V85$, and $V95$ speeds all decreased. At both the limit location and control location, the posted speed of the roadway facility had the most influence on all percentile speeds. As the ADT increased, speeds decreased at both limit location and control location, indicating that motorists may view increases in volume as a motivation to slow down.

TABLE 8 Speed Models as Reported by Jessen et al. (2001)

Models for minimum available sight distance (at point of limited stopping sight distance)	$V50 = 67.6 + 0.390(V_p) - 0.714(G_1) - 0.0017(T_{ADT})$	$R_a^2 = 0.57$
	$V85 = 86.8 + 0.297(V_p) - 0.614(G_1) - 0.00239(T_{ADT})$	$R_a^2 = 0.54$
	$V95 = 99.4 + 0.225(V_p) - 0.639(G_1) - 0.00240(T_{ADT})$	$R_a^2 = 0.57$
Models for control locations (non-limited stopping sight distance)	$V50 = 55.0 + 0.500(V_p) - 0.00148(T_{ADT})$	$R_a^2 = 0.44$
	$V85 = 72.1 + 0.432(V_p) - 0.0012(T_{ADT})$	$R_a^2 = 0.42$
	$V95 = 82.7 + 0.379(V_p) - 0.00200(T_{ADT})$	$R_a^2 = 0.40$

where
$V50$ = expected mean speed (km/h),
$V85$ = expected 85th percentile speed (km/h),
$V95$ = expected 95th percentile speed (km/h),
V_P = posted speed limit (km/h),
G_1 = approach grade (%), and
T_{ADT} = average daily traffic (valid for ADT ≤ 5,000 vpd).

Similar to the study by Jessen et al. (2001), Schurr et al. (2002) modeled $V50$, $V85$, and $V95$ free-flow speeds of passenger cars on horizontal curves in Nebraska. Speeds were observed at a control location (i.e., 600 ft before the point of curvature) and at the curve midpoint (i.e., midway between the points of curvature and tangency). Forty of the sites were used for model estimation and are summarized as shown in Table 9. The 40 sites were chosen to represent typical horizontal curvature on Nebraska highways with posted speeds of 55, 60, and 65 mph. Radii values ranged from 218 to 1,746 m (716 to 5,730 ft), while the majority of curves were between 350 to 1,746 m (1,146 to 5,730 ft). Profile grades varied between ± 4% with the majority of sites having grades of ± 2%. Regression models were estimated based on free-flow passenger car speeds in dry, daytime conditions. Separate models for speeds at the midpoint of a horizontal curve and speeds on the approach tangent are presented in Table 9. The authors concluded that certain elements of the horizontal curve may affect the speed of vehicles traversing them but the majority of drivers tend not to significantly reduce or increase their speeds when traveling from a tangent segment to horizontal curve when the radius is greater than or equal to 350 m (1,146 ft). The following conclusions were made about vehicles traveling at the midpoints of horizontal curves with a radius greater than or equal to 350 m (1,146 ft).

- As the deflection angle increases, speeds generally decrease.
- As the curve length increases, speeds generally increase.
- As the posted speed increases, $V50$ increases.
- As approach grade increases, $V85$ decreases.
- As ADT increases, $V95$ decreases.

TABLE 9 Speed models as reported by Schurr et al. (2002)

Models for passenger car speeds at curve midpoint:	$V50 = 67.4 - 0.1126\ \text{Delta} + 0.02243\ L + 0.276\ V_p$	$R_a^2 = 0.55$
	$V85 = 103.3 - 0.1253\ \text{Delta} + 0.0238\ L - 1.039\ G_1$	$R_a^2 = 0.46$
	$V95 = 113.9 - 0.122\ \text{Delta} + 0.0178\ L - 0.00184\ T_{\text{ADT}}$	$R_a^2 = 0.41$
Models for passenger car speeds at approach location:	$V50 = 51.7 + 0.508\ V_p$	$R_a^2 = 0.30$
	$V85 = 70.2 + 0.434\ V_p - 0.001307\ T_{\text{ADT}}$	$R_a^2 = 0.19$
	$V95 = 84.4 + 0.352\ V_p - 0.001399\ T_{\text{ADT}}$	$R_a^2 = 0.22$

where
- $V50$ = average speed of free-flow passenger cars (km/h);
- $V85$ = expected 85th percentile speed (km/h);
- $V95$ = expected 95th percentile speed (km/h);
- Delta = curve deflection angle (decimal degrees);
- L = arc length of curve (m);
- V_p = posted speed (km/h);
- G_1= approach grade (%);
- T_{ADT}= average daily traffic (valid for ADT $\leq$ 5,000 vpd); and
- SSD = Stopping sight distance.

FHWA Interactive Highway Safety Design Model (IHSDM) includes a Design Consistency Module (DCM) that estimates $V85$ for passenger cars. The algorithm used in the DCM is generally based on the results of an FHWA study published as Report FHWA-RD-99-171 (Fitzpatrick et al., 2000a). The main difference between the speed-profile model recommended in FHWA-RD-99-171 and the one currently used in the IHSDM DCM is that Equations 5 to 10 in FHWA-RD-99-171 were replaced by Equations 5A, 5B, and 6 as shown in posted speeds in Table 10 while Equations 1 through 4 (for horizontal curves on grades) remained the same. Acceleration and deceleration rates into and out of curves were recalibrated using the same data collected for FHWA-RD-99-171; new values were estimated, as shown in Table 11.

A limitation of the DCM operating speed profile model included in the 2003–2009 version of the IHSDM software is that it was calibrated using data from state-maintained highways with mainly 55-mph speed limits, although many of the curves studied had lower advisory speeds. The range of applicability of the DCM is constrained to prevent extrapolation beyond the range of data for which the underlying speed-prediction models were calibrated. As a result, the minimum speed predicted by the model is 60 km/h (37 mph). In order to expand the range of applicability of the DCM to lower speeds, FHWA collected additional data on lower-speed roadways in 2008 and calibrated the speed-prediction model for a wider range of conditions.

For the IHSDM 2010 Release, FHWA added speed profile models to the DCM for lower-speed highways (e.g., posted speeds 25 to 40 mph). To develop the models, FHWA obtained measurements of vehicle operating speeds at multiple locations along three specified highway routes in Virginia, California, and Idaho. Data were collected at over 80 sites, with posted speeds

TABLE 10 Speed Prediction Equations for Passenger Vehicles

No.	Alignment Condition	Equation
1.	Horizontal curve on grade: $G < -4\%$	$V_{85} = 102.10 - \frac{3077.13}{R}$
2.	Horizontal curve on grade: $-4\% \le G < 0\%$	$V_{85} = 105.98 - \frac{3709.90}{R}$
3.	Horizontal curve on grade: $0\% \le G < 4\%$	$V_{85} = 104.82 - \frac{3574.51}{R}$
4.	Horizontal curve on grade: $G \ge 4\%$	$V_{85} = 96.61 - \frac{2752.19}{R}$
5A.	Horizontal curve and vertical curve combined: vertical curve begins before midpoint of horizontal curve (PVC is before MP).	Calculate "effective grade" and use appropriate AC EQ 1–4.
5B.	Horizontal curve and vertical curve combined: vertical curve begins after midpoint of horizontal curve (PVC is after MP).	Use AC EQ 1–4 based on entry grade.
6.	Vertical curve on horizontal tangent	Calculate instantaneous grade and use TWOPAS to predict speed.

NOTE:

V_{85} = 85th percentile speed of passenger cars (km/h)
R = radius of curvature (m)
G = grade (%)
effective grade = the difference in elevation between the *PVC* (or *PVT*, for travel in the opposite direction) and midpoint of the vertical curve, divided by $L/2$, where L = length of vertical curve (m)

TABLE 11 Deceleration and Acceleration Rates

Deceleration Rate, *d* (m/s^2)	Alignment Condition		Acceleration Rate, *a* (m/s^2)
Radius, R(m) — d $R > 873$ — 0.05 $175 \le R \le 873$ — $-0.0008726 + \frac{37430}{R^2}$ $R < 175$ — 1.25	1–4	Horizontal curve on grade	Radius, R (m) — a $R > 436$ — 0.21 $250 \le R \le 436$ — 0.43 $R < 250$ — 0.54
Use rates for ACs 1–4, based on radius of horizontal curve	5A/B	Horizontal curve combined with vertical curve	Use rates for ACs 1–4, based on radius of horizontal curve
n/a	6	Vertical curve on horizontal tangent	n/a

of 30 and 35 mph. The resulting models included those for estimating speeds on horizontal tangents and curves, as noted in Table 12. For tangent speeds, separate models were developed for short tangents (tangent length < 150 ft) and for tangents 150 ft or longer. For short tangents, predictive variables were the posted speed of the highway and the radius of the curve preceding the tangent (model R^2 value = 0.49). For longer tangents, the posted speed, roadside hazard rating, and length of tangent were found to impact operating speeds (model R^2 value = 0.29). The curve speed model (R^2 value = 0.38) has the radius of curvature as a predictor variable. Tangent speeds are the maximum of the speeds predicted by the tangent model and the curve model; therefore, the final predicted tangent speed is not less than the predicted speed on the relevant curve). Acceleration and deceleration rate models were also developed, as noted in Table 13.

Adolini-Minnicino and Elefteriadou (2004) conducted research to expand the existing truck speed data available from FHWA-RD-99-171 (Fitzpatrick et al., 2000b) through simulation (using the TWOPAS simulation model) and then estimated regression models that predict truck speeds for a variety of horizontal and vertical alignments on two-lane rural highways. The authors noted that, when identifying alignments that may cause large speed differentials between cars and trucks, it is important that the differential is not underestimated by predicting the truck speed at a point that does not match the location of the true minimum truck speed. Therefore, models to predict truck speeds at 13 locations along the approach tangent, horizontal or vertical curve and departure tangent were developed.

TABLE 12 Models for Estimating 85th Percentile Speed on Tangents and Curves for Lower-Speed (Posted Speed 25 to 40 mph) Highways

Condition	Model
Tangent Speed: V_{T85} for $L_T < 150$ ft	Maximum of: 1) $V_{T85} = 35.15 + 0.26PS - \frac{1132}{R_{Preceding}}$ and 2) V_{C85} (for the curve preceding the tangent)
Tangent Speed: V_{T85} for $L_T \geq 150$ ft	Maximum of: 1) $V_{T85} = 26.04 + 0.53PS - 0.89RHR + 0.005L_T$ and 2) V_{C85} (for the curve following the tangent)
Curve Speed: V_{C85}	Minimum of: 1) $V_{C85} = 44.25 - \frac{1462}{R}$ and 2) CS_{UB}

where

V_{T85} = 85th percentile speed on tangent (mph);
LT = length of tangent (ft) (Note: use LT = 1,000 ft for tangent lengths greater than 1,000 ft);
PS = posted speed (mph);
$R_{Preceding}$ = radius of the preceding curve (ft);
RHR = roadside hazard rating (1 to 7);
V_{C85} = 85th percentile speed on curve (mph);
R = radius of the curve (ft); and
CS_{UB} = curve speed upper bound (posted speed plus 10 mph).

TABLE 13 Acceleration and Deceleration Rates used in IHSDM DCM for Lower-Speed Highways (Posted Speed 25 to 40 mph)

	Condition	Curve radius* (m)	Rates (m/s^2)
Deceleration	Tangent-to-curve	>873	0.05
		$175 \le R \le 873$	$-0.0008726 + 37430/R^2$
		<175	1.25
	Curve-to-tangent	All	0.05
	To a STOP condition		2.5
	To an end speed (>0 km/h)		1.25
Acceleration	Curve-to-tangent	>436	0.21
		$250 \le R \le 436$	0.43
		<250	0.54
	Tangent-to-curve	All	0.21
	From a STOP condition		1.54
	From a start speed (>0 km/h)		0.54

* For tangent-to-curve deceleration, the radius of the curve following the tangent is used. For curve-to-tangent acceleration, the radius of the curve preceding the tangent is used.

Horizontal Curves on Grades

Models for 13 locations on grades 0% to 5%:

- PC200 $V_{avg} = 107 - 8315/R$ $R^2 = 0.846$
- PC150 $V_{avg} = 103 - 7066/R$ $R^2 = 0.804$
- PC100 $V_{avg} = 103 - 7188/R$ $R^2 = 0.752$
- PC50 $V_{avg} = 95.8 - 4787/R$ $R^2 = 0.570$
- PC $V_{avg} = 94.7 - 2028/R - 1.98\ G_1 - 0.0140\ T_1$ $R^2 = 0.568$
- QP $V_{avg} = 95.6 - 3043/R - 1.79\ G_1 - 0.0110\ T_1$ $R^2 = 0.505$
- MP $V_{avg} = 95.6 - 3017/R - 1.85\ G_1 - 0.0111\ T_1$ $R^2 = 0.522$
- 3QP $V_{avg} = 96.1 - 4912/R$ $R^2 = 0.496$
- PT $V_{avg} = 96.0 - 3016/R - 2.23\ G_1 - 0.0120\ T_1$ $R^2 = 0.552$
- PT50 $V_{avg} = 98.0 - 5876/R$ $R^2 = 0.895$
- PT100 $V_{avg} = 91.1 - 3439/R$ $R^2 = 0.793$
- PT150 $V_{avg} = 90.9 - 3381/R$ $R^2 = 0.772$
- PT200 $V_{avg} = 90.9 - 3375/R$ $R^2 = 0.772$

Models for 13 locations on grades –5% to 0:

- PC200 $V_{avg} = 101 - 0.0277\ T_1$ $R^2 = 0.783$
- PC150 $V_{avg} = 95.3 - 3457/R$ $R^2 = 0.947$
- PC100 $V_{avg} = 95.1 - 3412/R$ $R^2 = 0.514$
- PC50 $V_{avg} = 94.8 - 3167/R$ $R^2 = 0.669$

- PC $V_{avg} = 95.1 - 3009/R + 1.27\ G_1$ $R^2 = 0.648$
- QP $V_{avg} = 94.9 - 2741/R + 1.80\ G_1$ $R^2 = 0.700$
- MP $V_{avg} = 94.7 - 2835/R + 1.64\ G_1$ $R^2 = 0.728$
- 3QP $V_{avg} = 95.0 - 2876/R + 2.14\ G_1$ $R^2 = 0.726$
- PT $V_{avg} = 94.4 - 2632/R + 2.11\ G_1$ $R^2 = 0.735$
- PT50 $V_{avg} = 96.2 - 3163/R + 3.12\ G_1$ $R^2 = 0.930$
- PT100 $V_{avg} = 96.0 - 3078/R + 3.16\ G_1$ $R^2 = 0.926$
- PT150 $V_{avg} = 95.9 - 3085/R + 3.05\ G_1$ $R^2 = 0.920$
- PT200 $V_{avg} = 96.0 - 3178/R + 2.82\ G_1$ $R^2 = 0.908$

Models for tangent and curve midpoint (MP):

- Tangent 0 to 5% $V_{avg} = 94.2 - 4088/R$ $R^2 = 0.822$
- MP, 0 to 5% $V_{avg} = 93.1 - 4051/R$ $R^2 = 0.839$
- Tangent > 5% $V_{avg} = 76.3 - 128\ e$ $R^2 = 0.263$
- MP > 5% $V_{avg} = 95.9 - 1439/R - 3.81\ G_1 - 0.0291\ T_1$ $R^2 = 0.626$
- Tangent ≤ 5% $V_{avg} = 93.7 - 3266/R$ $R^2 = 0.788$
- MP ≤ 5% $V_{avg} = 92.3 - 3157/R$ $R^2 = 0.966$
- Tangent –5% to 0 $V_{avg} = 99.7 - 4389/R + 1.71\ G_1$ $R^2 = 0.888$
- MP, –5% to 0 $V_{avg} = 95.3 - 4055/R$ $R^2 = 0.849$

Crest Vertical Curve on Horizontal Tangents (Tangent and Midpoint of VC)

- Tangent $V_{avg} = 98.2 - 0.299K - 1.03G_1 + 0.0260T_1$ $R^2 = 0.826$
- MP $V_{avg} = 90.0 - 0.0354\ L_v + 0.0463\ \mathrm{T}_1$ $R^2 = 0.727$

Sag Vertical Curve on Horizontal Tangents

No significant models found. Use desired speed.

Horizontal Curves Combined with Vertical Crest Curves

- Tangent $V_{avg} = 94.2 - 1248/R - 2.36\ G_1$ $R^2 = 0.774$
- MP $V_{avg} = 93.9 - 2331/R - 1.54\ G_1$ $R^2 = 0.786$

Horizontal Curves Combined with Vertical Sag Curves

- Tangent $V_{avg} = 98.3 - 2385/R - 0.046\ L_v$ $R^2 = 0.749$
- MP $V_{avg} = 94.6 - 3700/R + 111\ e - 0.0312\ L_v$ $R^2 = 0.895$

where

V_{avg} = mean speed (km/h);
R = radius of curvature (m);
G_1 = approach tangent grade (%);

T_1 = approach tangent length (m);
e = superelevation rate;
K = L/A = length of vertical curve / algebraic change in grade (%);
L_v = length of vertical curve (m);
PC# = number of meters before the point of curvature;
PT# = number of meters after the point of tangency;
QP = quarter point of curve;
MP = midpoint of curve; and
3QP = three-quarter point of curve.

A validation of TWOPAS was conducted as part of the study. The results of the validation demonstrated that TWOPAS is able to replicate mean truck speeds reasonably well, but it is not able to replicate the variance in truck speeds, nor truck speed distributions. Since TWOPAS is not able to replicate truck speed distributions, it is not able to reliably predict $V85$. Therefore, the models developed in the research predict mean truck speeds. However, the models can still be used to obtain a close approximation of $V85$ truck speeds if the distribution of speeds is known. The models can be summarized as follows.

Horizontal Curves on Grades

- Mean truck speeds are influenced by the radius of curve.
- Mean midpoint speeds on upgrades steeper than +5% also are influenced by the length and grade of the approach tangent.

Crest Vertical Curves on Horizontal Tangents

- Mean truck speeds are influenced by the length and grade of the approach tangent and the characteristics of the vertical curve.
- A *K*-value of 43 was not found to affect models for trucks, implying that this value should not be used to differentiate limited and no-limited sight distance crest curves for trucks.

Horizontal Curves Combined with Vertical Crest Curves

- Mean truck speeds are influenced by the radius of curve and grade of the approach tangent.
- The effects of the horizontal curve overshadow the effects of the vertical curve.

Horizontal Curves Combined with Vertical Sag Curves

- Mean truck speeds are influenced by the radius of horizontal curve and length of the vertical curve.
- Mean MP speeds are also influenced by the superelevation of the curve.

Figueroa, Medina, and Tarko (2005) collected free-flow speeds and geometric characteristics at 158 locations on two-lane rural highway segments in Indiana. The locations were at various distances before, after, and within horizontal curves. Locations also included intersections and vertical curves. The average number of speed observations was 360, with at least 100 observations at each spot. Data from horizontal curves with radii greater than 1,700 ft were combined with data from tangent sections for modeling purposes. Panel data were created by computing the 5th through the 95th percentile speeds at each location and multiplying the possible explanatory variables by the corresponding standard normal value (z_P) for that percentile (assuming a normal distribution). This resulted in 19 percentile speeds (i.e., observations) at each site. The general form of the panel data (PD) model estimated using OLS (called the OLS-PD estimator) was as follows:

$$V_{ip} = \sum_j a_j * X_{ij} + \sum_k b_k * \left(Z_p * X_{ik}\right) + \varepsilon$$

where

V_{ip} = the pth percentile speed at site i;
X_{ij}, X_{ik}= exogenous variables affecting mean speed and standard deviation of speeds, respectively;
Z_p = the standard normal value for the pth percentile;
a_j, b_k = parameters quantifying the relationship between X_{ij}, X_{ik} and V_{ip}; and
ε = random disturbance.

Data from 85 spot locations on tangents and 14 spot locations on horizontal curves were used to estimate the models that are presented in Table 14 and Table 15.

Ten highway variables, six of them functioning as both mean speed and speed dispersion factors, were identified as speed factors on tangent segments. Four highway and curve variables, two of them functioning as both mean speed and speed dispersion factors, were identified as speed factors on horizontal curves. The developed free-flow speed models have the same prediction capabilities as traditional ordinary-least-squares models estimated for specific percentile speeds. The advantages of the models include predicting any user-specified percentile, involving more highway characteristics as speed factors than traditional regression models, and separating the impacts on mean speed from the impacts on speed dispersion. Another contribution is that the impact of the cross-section dimensions is present in the tangent speed model (Figueroa, Medina, and Tarko, 2005).

Schurr et al. (2005) also developed models that describe design speed profiles of vehicles traversing horizontal curves on approaches to stop-controlled intersections on two-lane two-way rural highways. Speed profiles were developed with the use of data from 15 study sites in Nebraska. Of those sites, three were on tangent approaches to stop-controlled intersections, and were used to determine the vehicle speed profiles on approaches to a stop-controlled intersection without the influence of a horizontal curve. The other 12 sites contained a simple curve, reverse curve, or compound curve in the roadway alignment as it approached a stop.

All types of sections were selected to determine if and how horizontal curves influenced the vehicle speed profiles. Power regression models were estimated to provide speed prediction equations for developing speed profiles (see Table 16). Separate profiles were created for passenger cars and heavy vehicles because speeds were significantly different. The heavy vehicle data were separated further for alignments with and without horizontal curvature. It was

TABLE 14 OLS-PD Speed Model for Tangent Segments (Figueroa, Medina, and Tarko, 2005)

Variable	Coefficient	Standard Error	*t*-value	*p*-value
Mean speed factors				
Constant	57.1372	0.6019	94.93	< 0.0001
Percent trucks	–0.0710	0.0109	–6.54	< 0.0001
50-mph posted speed limit indicator	–3.0818	0.1404	–21.96	< 0.0001
Highway grade	–0.1307	0.0248	–5.26	< 0.0001
Residential development indicator	–1.0338	0.1368	–7.56	< 0.0001
Sight distance	2.38×10^{-3}	6.326×10^{-4}	3.76	0.0002
Sight distance squared	-1.67×10^{-6}	2.566×10^{-7}	–6.51	< 0.0001
Intersection indicator	–0.4216	0.1234	–3.42	0.0006
Pavement width	0.0401	0.0095	4.23	< 0.0001
Gravel shoulder width	0.3941	0.0329	12.10	< 0.0001
Untreated shoulder width	0.0544	0.0047	11.50	< 0.0001
Flat curve indicator	–2.2329	0.1577	14.16	< 0.0001
Speed dispersion factors				
Constant	5.9816	0.2786	21.47	< 0.0001
50-mph posted speed limit indicator	1.4280	0.1498	9.53	< 0.0001
Highway grade	0.0608	0.0283	2.15	0.0319
Intersection indicator	0.2917	0.1389	2.10	0.0359
Pavement width	–0.0382	0.0083	–4.62	< 0.0001
Roadside clear zone	–0.0118	0.0048	–2.46	0.0140

NOTES: Adjusted R^2 = 0.8442; RMSE = 2.117

TABLE 15 OLS-PD Speed Model for Horizontal Curves (Figueroa, Medina, and Tarko, 2005)

Variable	Coefficient	Standard Error	*t*-value	*p*-value
Mean speed factors				
Constant	47.6639	0.7038	67.73	< 0.0001
Sight distance	3.4400×10^{-3}	3.8581×10^{-4}	8.91	< 0.0001
Residential development indicator	–2.6388	0.3777	–6.99	< 0.0001
Degree of curve	–2.5409	0.0722	–35.17	< 0.0001
Superelevation rate	7.9535	0.2564	31.02	< 0.0001
Superelevation rate squared	–0.6239	0.0192	–32.57	< 0.0001
Speed dispersion factors				
Constant	4.1576	0.4049	10.27	< 0.0001
Degree of curve	0.2358	0.0670	3.52	0.0005
Superelevation rate	–0.1987	0.0679	–2.92	0.0038

NOTES: Adjusted R^2 = 0.9322; RMSE = 1.757

TABLE 16 Speed Models Developed by Schurr et al. (2005)

Vehicle Type	Model	R_a^2	Correlation Coefficient	Significance
Passenger cars	$V95 = 10.42\ d^{0.250}$	0.896	0.947	95th percent confidence level
Heavy vehicles at curve sites	$V95 = 12.0\ d^{0.219}$	0.844	0.919	95th percent confidence level
Heavy vehicles at tangent sites	$V95 = 14.6\ d^{0.197}$	0.777	0.885	95th percent confidence level

NOTE: $V95$ = 95th percentile free-flow speed of vehicles in dry, daytime conditions (mph); d = distance to stop (ft)

concluded that the posted speed limit, median type, presence of rumble bars, roadway surface condition, and degree of rutting did not significantly affect the vehicle speed profiles at these sites at a 95% confidence level.

When comparing speed models, it was found that the intercepts of the regression lines for approaches with and without horizontal curves were significantly different in the case of heavy vehicles. The speed of heavy vehicles on tangent approaches was generally about 8 mph higher than on sites that exhibited horizontal curvature. The rate of deceleration remained almost the same on all approaches to intersections, except near the stop. For passenger cars, there was no statistically significant difference between curved and tangent alignments. The following models were developed.

Misaghi and Hassan (2005) conducted research to estimate operating speed and speed differential models for two-lane, rural highways. The scope of the study was limited to daylight conditions and dry pavement. The study area included different highway classes and a total of 20 curves with various horizontal curve radii, lengths, and vertical grades. Locations were selected on four different two-lane rural highways in Ontario, Canada. Speed data were collected for almost 24 h on each curve. Observations on three points per curve (approach tangent, curve midpoint, and departure tangent) were collected for all curves. The authors estimated two sets of models, 85th percentile speed models and speed differential models, using speed data collected with traffic counters/classifiers on the 20 curves.

Only the radius of horizontal curve, or the square of horizontal curve radius, were found to statistically significant predictors of 85th percentile speeds when 85th percentile speed was modeled as a point speed measure with OLS regression (similar to most of the studies previously described in this chapter). The speed on curves for passenger cars was modeled as

$$V85_{MC} = 91.85 + 9.81 \times 10^{-3}\, R \qquad R^2 = 0.464$$
$$V85_{MC} = 94.30 + 8.67 \times 10^{-6}\, R^2 \qquad R^2 = 0.524$$

where

$V85_{MC}$ = 85th percentile speed at midpoint of horizontal curve for passenger cars and light trucks (km/h) and
R = radius of curve (m).

Additional statistically significant predictors were found for the 85th percentile speed differential from a tangent to a curve. The speed differential models were reported as

$$\text{Delta}V85 = -83.63 + 0.93\ V_T + e^{(-8.93\ +\ 3507.10/R)} \qquad R^2 = 0.640$$

$$\text{Delta}V85 = -198.74 + 21.42\ (\sqrt{V_T}) + 0.11\ \text{DFC} - 4.55\ \text{SW} - 5.36\ (\text{curve\_dir}) + 1.30\ G + 4.22\ (\text{drv\_flag}) \qquad R^2 = 0.889$$

where

Delta*V*85 = 85th percentile speed differential, i.e., the differential speed not exceeded by 85% of the drivers traveling under free-flow conditions, for passenger cars and light trucks;
V_T = speed on approach tangent (km/h);
DFC = deflection angle of circular curve (degrees);
G = average longitudinal grade from approach tangent (AT) to midpoint of curve (MC);
SW = shoulder width (m);
curve_dir = curve direction flag (left = 1, right = 0); and
drv_flag = driveway flag (intersection on curve = 1, otherwise = 0).

Speed differentials were calculated for each individual vehicle in the stream of traffic. For each lane of traffic, the speed change from approach tangent to the curve midpoint was investigated. Having all the individual speed differential values, the 85th percentile value of the speed differentials was calculated for each lane of traffic of each curve and referred to as Delta*V*85 (Misaghi, 2003). Delta*V*85 is fundamentally different from the speed differential resulting from subtraction of operating speeds on two successive elements (Delta$V85 = V85_i - V85_{i-1}$). Based on the observations of this study and similar to the study by McFadden and Elefteriadou (2000), it was concluded that the simple subtraction of operating speeds at the approach tangent and the middle of the curve underestimates the real values of speed differential (Misaghi and Hassan, 2005).

Recently, Nie and Hassan (2007) conducted a field experiment to analyze driver speed behavior on the most common road types in Eastern Ontario, Canada. As shown in Table 17, speed prediction models were estimated using actual driving data for two-lane rural highways and urban/suburban roads. The models considered driver speed behavior when negotiating horizontal curves. A total of 10 horizontal curves were available for model development on two-lane rural highways. On average, each study curve had 25 free-flow speed observations. The presence of an intersection was not considered and the posted speed was not included as an independent variable because all sites had a speed limit of 80 km/h. A series of speed and speed differential models were established using data from all 10 curves. On four “non-independent” curves with short tangents, speed increases were observed from the approach tangent into the curve. Regression analysis was carried out a second time exclusively for the remaining six independent curves, and models were estimated for independent curves alone on two-lane rural highways.

TABLE 17 Speed Models Reported by Nie and Hassan (2007)

Recommended Models for Two-Lane Rural Highways				
No.	Model	df	Adjusted R^2	SEE
All Curves				
1	$V85\_AT = 81.782 + 0.086LAT$	8	0.661	4.24
2	$V85\_BC = 108.132 - 0.090CCR$	8	0.714	4.03
3	$V85\_MC = 108.357 - 0.097CCR$	8	0.860	2.83
4	$V85\_EC = 102.238 - 0.092CCR + 0.039LDT$	7	0.938	1.83
5	$V85\_DT = 78.690 + 0.00001127R^2 + 0.066LDT$	7	0.857	2.95
6	$85MSR = 17.857 - 0.080LDT + 7.324DFC$	7	0.729	4.23
7	$85MSI = -0.410 + 0.078LDT$	8	0.715	3.42
Curves with Independent Approach Tangent				
8	$V85\_BC = 30.563 + 10.582\ln(R)$	4	0.776	3.14
9	$V85\_MC = 110.386 - 6856.213(1/R)$	4	0.908	2.48
10	$V85\_EC = 111.404 - 7360.698(1/R)$	4	0.868	3.24
11	$V85\_DT = 76.486 + 0.127LDT$	4	0.767	5.03
12	$\Delta V85 = -5.326 + 0.073CCR$	4	0.574	4.22
13	$\Delta 85V = -4.540 + 0.088CCR$	4	0.749	3.56
14	$85MSR = -0.658 + 0.107CCR$	4	0.760	4.22

NOTE: df = degrees of freedom; adjusted R^2 = adjusted coefficient of determination;
SEE = standard error of the estimate.

where

$V85\_AT$ = 85th percentile speed on the approach tangent (AT) measured at the point AT + 100 (for curves with AT > 100 m) or at the start of the AT (otherwise);

$V85\_DT$ = 85th percentile speed on the departure tangent (DT) measured at the point DT + 100 (for curves with DT > 100 m) or at the end of the DT (otherwise);

$V85\_BC$, $V85\_MC$ and $V85\_EC$ = 85th percentile speed at the beginning, MP, and end of the circular curve, respectively;

$\Delta V85$ = Operating speed differential ($V85\_AT - V85\_MC$);

$\Delta 85V$ = Operating speed differential calculated as the 85th percentile value of individual speed differentials. Each individual speed differential in the distribution is taken as ($V\_AT - V\_MC$) for an individual driver;

85MSR = Operating speed differential calculated as the 85th percentile value of individual maximum speed reductions. Each individual maximum speed reduction in the distribution is taken as the difference between the maximum speed on the approach tangent and the minimum speed on the curve for an individual driver;

85MSI = Operating speed differential calculated as the 85th percentile value of individual maximum speed increases. Each individual maximum speed increase in the distribution is taken as the difference between the maximum speed on the DT and the minimum speed on the curve for an individual driver;

LAT = length of approach tangent (m);

CCR = curvature change rate;

LDT = length of departure tangent (m);

R = radius of curve (m); and

DFC = curve deflection angle (degrees).

The authors concluded that driver speed selection on two-lane rural highways is highly correlated to the geometric features. Tangent speeds are positively correlated with tangent length. Operating speeds on the curved section are correlated to curve parameters CCR, ln(R), or $1/R$. Operating speeds at the end of a circular curve can also be affected by the departure tangent length. The speed changes when the vehicle traverses a horizontal curve are governed by both curve parameters and the lengths of the adjacent tangents. The authors note that this finding supports the importance of the relation between the adjacent elements on a roadway as stated in the work by Lamm et al. (1988) and Lamm and Smith (1994). The inclusion of the departure tangent length (LDT) in the speed differential estimation (e.g., 85MSR) implies that drivers are more cautious when negotiating multiple curves in close succession than they are on isolated curves. According to the authors, speed differential in terms of $\Delta V85$ is again proven to underestimate the speed reduction from the tangent to the curve, as demonstrated by other researchers (e.g., Hirshe, 1987; McFadden and Elefteriadou, 2000; and Misaghi and Hassan, 2005). This study also showed that drivers do not maintain a constant speed, and deceleration or acceleration may take place on the curved section. The authors stated that the models should be used with some degree of caution because of the relatively small sample size in terms of number of sites and number of observations at each site.

Fambro et al. (2000) collected free-flow speed data on 41 vertical tangent-curve combinations in three states: Washington, Texas, and Illinois. Roadway cross sections included multilane roadways, two-lane roadways with shoulders of 1.8 m or wider (categorized as two-lane-with-shoulder), and two-lane roadways with shoulders less than 1.8 m (categorized as two-lane-without-shoulder). Speeds were collected on the tangents and at the point of minimum sight distance, i.e., on the vertical curves just before the point of vertical intersection. Data were collected for a minimum of 4 h or 100 vehicles. The following model for $V85$ on vertical curves was reported for two-lane roads without shoulders as follows:

$$V85 = 72.5 + 0.3(\text{IDS}) \qquad R^2 = 0.48$$

where

$V85$ = expected 85th percentile speed on vertical curves (km/h) and
IDS = inferred design speed (km/h).

The study results indicated that both the 85th percentile and the mean operating speeds were well above the design speeds of the crest vertical curves in the range of conditions studied. Data from all the roadways studied suggest that the lower the design speed the larger the difference between $V85$ and design speed. In addition, available sight distance appears to influence mean speed reductions between the control and crest sections. As available sight distance decreases, the mean speed reductions between the control and crest sections tend to increase. However, the reduction in speed is less than that suggested by current AASHTO criteria. For two-lane roadways without shoulders, $V85$ can be predicted as a function of available sight distance at crest vertical curves. For the range of conditions studied, traffic volume and roadway type appeared to have little influence on mean speed reductions between both control and crest sections.

MULTILANE RURAL HIGHWAYS, URBAN STREETS, AND FREEWAYS

The *Highway Capacity Manual* allows for several assumptions to be made concerning the relationship between the posted speed limit and operating speed when analyzing rural multilane roadways. When the posted speed limit is 40 or 45 mph, it is assumed that the base free-flow speed under ideal conditions exceeds the speed limit by 7 mph. When the posted speed is 50 or 55 mph, it is assumed that the base free-flow speed exceeds the posted speed limit by 5 mph. This assumption can be exercised when a field study cannot be performed.

Dixon et al. (1999) completed a study to determine the relationship between the posted speed limit and operating speed because of the repeal of the 55-mph national speed limit. The effects of roadway geometry were considered to make adjustments to the base free-flow speed. The Georgia Department of Transportation (DOT) collected speed data at 12 rural multilane stationary counting locations prior to the repeal of the national speed limit. These counting locations were used to determine the effects of the posted speed limit being raised to 65 mph. Speed and volume data were collected for continuous 24-h periods by Georgia DOT personnel both before and after raising the speed limit. Geometric conditions, such as grade, horizontal geometry, proximity to adjacent intersections, and number of access points were determined from as-built drawings as well as highway video photo logs. It was found that the observed mean speed increased by 3.2 mph when the posted speed increased from 55 mph to 65 mph. However, it was noted that the data were collected only a few months after the increase in posted speed. It was hypothesized that given more time to adjust to the higher regulatory conditions, mean speeds will likely continue to increase. A general negative trend was found when considering the relationship between free-flow speed and access density. However, the result was not statistically significant. It was also shown that steeper positive grades limited the change in the standard error in speeds from the before to after periods. The relationship between observed free-flow speeds and traffic volume and heavy vehicle percentage were nominal. It was recommended that future speed research on multilane highways attempt to quantify the relationship between several geometric features (i.e., steep vertical grade, heavy vehicle percentages, or high traffic volumes) and free-flow operating speed. It was also recommended that less-than-ideal conditions are studied to determine the effects of offset to lateral obstructions, narrow lanes, or absence of medians on free-flow operating speeds.

Ali et al. (2007) also studied the interrelationship between the free-flow speed, posted speed limit, and geometric design variables along 35 four-lane urban streets in Fairfax County, Virginia. The study sites consisted of facilities that had an average intersection density of less than 2 mi and posted speed limits that ranged from 35 to 45 mph. Free-flow speeds were collected for passenger cars only. Spot speed data were collected using radar guns at mid-block locations on horizontal and vertical tangent sections. Vehicles traveling in free-flow conditions were considered to have time headways of at least 7 s and tailways of at least 4 s. The number of speeds collected at each site ranged from 26 to 61, which led to a total of 1,742 spot speeds. Roadway geometry data was collected using a combination of field measurement and a geographic information system (GIS) database. The relationship between free-flow speed and posted speed, lane width, median type, median width, access density, and adjacent land use was explored. Segment length was also considered in the analysis; it was defined as the ratio of the study segment length to the maximum signal spacing. Correlation analysis showed that posted speed, median width, and segment length had a significant effect on free-flow speed on urban streets. The mean and 85th percentile aggregated free-flow speeds were modeled using linear regression. The models specified were

$$FFS_{mean} = 39.3 + 8.6\ PS_{45} + 3.7\ PS_{40} \qquad R^2 = 0.76$$
$$FFS_{85} = 42.3 + 10.4\ PS_{45} + 3.8\ PS_{40} \qquad R^2 = 0.77$$

where

FFS_{mean} = the mean free-flow speed (mph);
FFS_{85} = the 85th percentile free-flow speed (mph);
PS_{45} = posted speed (1 if posted speed is 45; 0 otherwise, baseline 35); and
PS_{40} = posted speed (1 if posted speed is 40; 0 otherwise, baseline 35).

Regression models that included the median type and segment length variables were also specified. These models are

$$FFS_{mean} = 37.4 + 6.8\ PS_{45} + 2.6\ PS_{40} + 13.5\ SL \qquad R^2 = 0.87$$
$$FFS_{85} = 37.4 + 8.0\ PS_{45} + 2.1\ PS_{40} + 3.6\ MT + 13\ SL \qquad R^2 = 0.86$$

where

SL = segment length ratio and
MT = median type (1 if divided or two-way left-turn lane; 0 if no median).

It was also concluded that while there was no significant relationship found between lane width and free-flow speed, future work should consider this variable further. Analysis of the data showed that both mean and 85th percentile speeds were higher than the posted speed limit and that the observed mean and 85th percentile free-flow speeds increase as the posted speed limit increases. Additionally, the mean and 85th percentile speeds were higher on sites with medians when compared to sites without medians.

Figueroa and Tarko (2004) studied the relationship between various roadway and roadside design features and operating speeds on four-lane roadways in Indiana. A variety of statistical modeling approaches were considered including OLS regression and PD analysis. The objective of the research was to develop guidance to assist designers in creating environments where the operating speed is in harmony with the posted speed limit. Data collection on the roadways included horizontal and vertical alignment data. However, an emphasis was placed on the diversification of the cross section dimensions due to the uniform nature of horizontal and vertical curvature on multilane roadways. Emphasis was also placed on collecting data along suburban roadway segments as opposed to rural segments (freeways were not considered). Multilane segments with low crash rates were chosen. Segments were also chosen to avoid close proximity to traffic signals. The following data elements were collected in the field.

- General: terrain, rural versus suburban area type, pavement surface, and posted speed limit;
- Access density: intersection density, driveway density, median opening density, and presence of residential or commercial developments;

- Tangents: grade, sight distance, cross-section dimensions, and roadside obstructions;
- Roadside features: obstruction type and presence of auxiliary lane or sidewalks;
- Intersections: intersection type and presence of channelization or auxiliary lanes;
- Horizontal curves: radius, maximum superelevation rate, and length of curve; and
- Distance to the beginning of horizontal curves and the middle of intersections, if present.

Free-flow speeds were collected using a Laser Atlanta laser gun and rubber tubes connected to a PEEK ADR-2000 traffic classifier at 50 sites. Free-flow speeds were considered to have time headways of 5 s or more. At least 100 passenger car speeds were collected at each site. Emergency vehicles, motorcycles, and turning vehicles were excluded from the dataset. A panel data model specification was used to estimate operating speed. The model for four-lane highways was specified as

$$\begin{aligned} V_p = {} & 53.884 - 4.753\ \mathrm{PSL}_{50} - 5.481\ \mathrm{PSL}_{45} - 7.432\ \mathrm{PSL}_{40} + 2.045\ \mathrm{RUR} + 0.00087\ \mathrm{SD} \\ & - 0.279\ \mathrm{INTD} - 0.023\ \mathrm{DRWD} + 1.732\ \mathrm{PS} + 0.02\ \mathrm{ECLR} + 0.046\ \mathrm{ICLR} - 2.102\ \mathrm{RAIL} \\ & - 1.193\ \mathrm{DITCH} + 6.051\ Z_p - 0.496\ (Z_p \times \mathrm{PSL}_{45\text{-}40}) - 0.585\ (Z_p \times \mathrm{RUR}) \\ & - 0.00042\ (Z_p \times \mathrm{SD}) - 0.011\ (Z_p \times \mathrm{ECLR}) - 0.430\ (Z_p \times \mathrm{TWLTL}) \qquad R^2 = 0.86 \end{aligned}$$

where

PSL_{50}	= 1 if the posted speed is 50 mph; 0 otherwise (baseline 55);
PSL_{45}	= 1 if the posted speed is 45 mph; 0 otherwise (baseline 55);
PSL_{40}	= 1 if the posted speed is 40 mph; 0 otherwise (baseline 55);
$\mathrm{PSL}_{45\text{-}40}$	= 1 if the posted speed is 45 or 40 mph; 0 otherwise;
RUR	= 1 if the segment is in a rural area; 0 otherwise;
SD	= sight distance (ft);
INTD	= intersection density (number of intersections per mile);
DRWD	= driveway density (number of adjacent driveways per mile);
PS	= 1 if the highway segment has a paved shoulder; 0 otherwise;
ECLR	= external clear zone, distance from edge of traveled way to roadside obstruction (ft);
ICLR	= internal clear zone, distance from internal edge of traveled way to inside edge of opposing traveled way or median barrier face (ft);
RAIL	= 1 if a guardrail is located 20 ft or less from the edge of the traveled way, 0 otherwise;
DITCH	= 1 if the ditch is located 20 ft or less from the edge of the traveled way, 0 otherwise;
TWLTL	= 1 if a two-way left-turn median lane is present, 0 otherwise; and
Z_p	= standardized normal variable corresponding to a selected percentile speed.

A random effects model was also estimated. The best random effects model specification was

$$\begin{aligned} V_p = {} & 54.027 - 4.764\ \mathrm{PSL}_{50} - 4.942\ \mathrm{PSL}_{45} - 6.509\ \mathrm{PSL}_{40} + 1.652\ \mathrm{RUR} + 0.00128\ \mathrm{SD} \\ & - 0.320\ \mathrm{INTD} + 0.034\ \mathrm{ECLR} + 0.056\ \mathrm{ICLR} + 5.899\ Z_p - 0.464\ (Z_p \times \mathrm{PSL}_{45\text{-}40}) \\ & - 0.464\ (Z_p \times \mathrm{RUR}) - 0.00048\ (Z_p \times \mathrm{SD}) - 0.00422\ (Z_p \times \mathrm{CLR}) - 0.477\ (Z_p \times \mathrm{TWLTL}) \end{aligned}$$

where CLR = total clear zone, includes the median width and external clear zone (ft).

The random effects model was suggested for implementation. This model showed that increasing the posted speed limit resulted in higher operating speeds. It also showed that speeds are higher in rural areas and when sight distance and external and internal clear zones are increased. Speeds decrease when intersection density increases. It also showed that the speed dispersion decreases by setting low speed limits, increasing sight distance, increasing total clear zone, and the presence of a two-way left-turn lane.

Fitzpatrick et al. (1997) collected data in the right lane of suburban roadways along 14 horizontal curves and 10 vertical curves in Texas. The objective was to relate inferred design speed to the 85th percentile operating speed. The curve radius and approach access density were collected for each horizontal and vertical curve. Spot speeds were collected near the MP of horizontal curves and at the point of minimum available sight distance on crest vertical curves. Only data for free-flow passenger cars, pickup trucks, and vans were considered in this study. Vehicles were considered to be free-flow if they had time headways of 5 s or more. Speeds were measured for through vehicles in the right lane only using both laser and radar guns. A total of 150 spot speeds were collected at each site. Spot speeds were collected on the approach tangent of each horizontal curve. Approach tangent speed was predicted as a function of the approach access density. The models were developed using OLS regression.

$$V85_{\mathrm{tan}} = 74.91 + 22.29/\mathrm{AD} \qquad R^2 = 0.71$$
$$V85_{\mathrm{curve}} = 43.5 + 0.38(\mathrm{IDS}) \qquad R^2 = 0.83$$
$$V85_{\mathrm{curve}} = 56.34 + 0.808R^{0.5} + 9.34/\mathrm{AD}$$

where

$V85_{\mathrm{tan}}$ = the 85th percentile approach tangent speed (km/h);
$V85_{\mathrm{curve}}$ = 85th percentile curve speed (km/h);
AD= approach access density (number of access points per km);
IDS = inferred design speed (km/h); and
R = curve radius (m).

Operating speed models for vertical curves were also considered using access density as the explanatory variable. A low proportion of the variation in the 85th-percentile operating speed was explained by this model. A regression model to predict the 85th-percentile operating speeds on vertical curves as a function of the inferred design speed was also estimated as

$$V85_{\mathrm{curve}} = 39.51 + 0.556(\mathrm{IDS}) \qquad R^2 = 0.56$$

The authors, however, noted that the sample size was too small to determine if the model was widely transferable. The results of this research showed that operating speeds generally exceed the inferred design speed on roadways with low inferred design speeds (less than 70 km/h for horizontal curves, and 90 km/h for vertical curves).

Fitzpatrick et al. (2003) explored speed relationships and agency practices related to speed. The purpose of this study was to investigate how the design speed of a roadway is selected and how design speed, operating speed, and posted speed are related. Roadway geometric elements that are related to the design speed were identified and critically reviewed. An operating speed model was also developed to facilitate discussion of practical alternatives to the design speed concept. The research team modeled operating speeds at 78 suburban/urban sites in Arkansas, Missouri, Tennessee, Oregon, Massachusetts, and Texas. Free-flow speeds were obtained using time headway of 5 s or more and a tailway of 3 s or more. Several site characteristics, including cross section elements, traffic control devices, roadside, and alignment features were collected for use as predictor variables in an operating speed model. Only the posted speed limit was found to be a statistically significant predictor of 85th percentile operating speed on urban–suburban arterials. Segment access density was included as a predictor of operating speed in a second model, but it was not statistically significant at the 95% confidence level. The estimated models were

$$V85 = 7.675 + 0.98\text{PSL} \qquad R^2 = 0.90$$
$$V85 = 16.089 + 0.831\text{PSL} - 0.054\text{AD} \qquad R^2 = 0.92$$

where

$V85$ = 85th-percentile operating speed (mph),
PSL = posted speed limit (mph), and
AD = access density (pts/mi).

Nie and Hassan (2007) modeled operating speeds on horizontal curves using data collected from a road experiment involving volunteer drivers and a test vehicle in Ontario, Canada. Continuous speed data were collected using instrumentation within the test vehicle. Geometric features were determined using GIS software. Driver speed trends were modeled using ordinary least squares regression. Operating speeds along a horizontal curve were modeled, as well as speed differential values when approaching and departing the curve. The test route covered seven roads that included an urban freeway, two-lane rural highways, a rural freeway, and urban–suburban roads. A passenger minivan was outfitted with equipment to collect instantaneous speed, lateral, and longitudinal acceleration, and the positions of the fuel pedal, brake, and steering wheel. The vehicle's path was monitored using a Global Positioning System (GPS) receiver. Thirty participants were recruited to drive the test vehicle using their normal driving habits. The drivers were not aware of the beginning and ending points of the experiment. A lidar gun was used to determine the following distance of the test driver to a leading vehicle. Free-flow conditions were defined using minimum time headways of 5 s. Several roadway variables were found to be significant predictors of speed on urban and suburban roadways. As shown in Table 18, models were specified for free-flow speeds only and for all speeds. It was found that including only free-flow speeds on urban roadways significantly reduced the sample size and therefore may not be practical since free-flow conditions occur less frequently.

TABLE 18 Recommended Speed Models by Nie and Hassan (2007)

No.	Model (km/h)	df	R^2	SEE
Free-flow speeds only and accounting for intersections				
1	$V85_{AT} = 23.686 + 0.807V_p$	5	0.698	5.48
2	$V85_{BC} = 60.643 – 28.819DFC + 0.540V_p$	4	0.872	4.07
3	$V85_{MC} = 54.738 – 29.031DFC + 0.618V_p$	4	0.931	3.12
4	$V85_{EC} = 55.220 – 30.113\text{-}DFC + 0.613V_p$	4	0.926	3.30
5	$V85_{DT} = 102.171 – 37.630DFC$	5	0.774	4.71
6	$_DV85 = –10.671 + 17.309DFC$	5	0.868	1.58
7	$_DV85 = –6.991 + 15.963\text{-}DFC – 0.0118LDT$	4	0.966	0.80
8	$_{D85}V = –4.386 + 14.130DFC$	5	0.733	1.96
9	85MSR = 24.401 – 0.800LDT	5	0.703	3.80
10	85MSR = 13.605 – 0.070LDT + 13.352DFC	4	0.919	1.98
All speeds accounting for intersections				
1	$V85_{AT} = 53.432 +0.483V_p – 9.085INT – 3.806R_1/R_2$	8	0.873	2.48
2	$V85_{BC} = 58.397 – 6.691R_1/R_2 – 10.741INT + 0.458V_p$	8	0.927	2.32
3	$V85_{MC} = 53.358 – 6.215R_1/R_2 + 0.499V_p – 8.262INT$	8	0.841	3.43
4	$V85_{EC} = 52.675 – 0.026CCR + 0.400V_p$	9	0.587	5.63
5	$V85_{DT} = 95.586 – 31.126DFC$	10	0.552	5.80
6	$85MSI = 30.441 + 10.886INT – 0.271V_p – 0.029LAT$	8	0.803	2.43
All speeds without accounting for intersections				
1	$V85_{AT} = 49.709 + 0.546V_p – 4.121R_1/R_2$	5	0.936	2.02
2	$V85_{BC} = -2.239 + 8.080 \ln R + 0.486V_p$	5	0.908	2.99
3	$V85_{MC} = 58.589 – 24.797DFC + 0.522V_p$	5	0.839	4.00
4	$V85_{MC} = 53.710 – 10.464DFC + 0.559V_p – 4.495R_1/R_2$	4	0.992	0.87
5	$V85_{EC} = 60.494 – 14.150DFC + 0.435V_p – 0.018CCR$	4	0.938	2.35
6	$V85_{DT} = 99.525 – 34.274DFC$	6	0.753	4.23
7	$_DV85 = –6.280 + 9.418DFC\ 0.023LAT$	5	0.815	1.48
8	$_DV85 = –2.885 + 5.541DFC + 0.029LAT – 5.240 \times 10^{–6} R^2$	4	0.921	0.97
9	$_{D85}V = –5.046 + 16.434DFC$	6	0.743	2.08
10	$_{D85}V = –5.405 +13.088DFC + 0.023LAT$	5	0.871	1.48
11	85MSR = 1.559 + 0.100LAT	6	0.764	3.98

where

$V85AT$ = 85th percentile speed 100 m before the beginning of a horizontal curve;
$V85BC$ = 85th percentile speed at the beginning of a horizontal curve;
$V85MC$ = 85th percentile speed at the middle of a horizontal curve;
$V85EC$ = 85th percentile speed at the end of a horizontal curve;
$V85DT$ = 85th percentile speed 100 m after the end of a horizontal curve;
$_DV85$ = Difference between $V85AT$ and $V85MC$;
$_{D85}V$ = 85th percentile value of individual speed differences between $V85AT$ and $V85MC$;
85MSR = 85th percentile value of individual maximum speed reductions;
85MSI = 85th percentile value of individual maximum speed increase (from the middle of a horizontal curve to the departure tangent);
V_p = posted speed;
DFC = curve deflection angle;
LDT = length of departure tangent;
INT = presence of intersection (1 if intersection present; 0 otherwise);
R = horizontal curve radius;
CCR = curvature change rate; and
LAT = length of approach tangent.

It was concluded that posted speed has a positive association with operating speed on urban/suburban roadways. It was also found that the presence of intersections has a negative association with operating speed. Curve parameters were also found to be associated with operating speed. The ratio between the radius of the curve and radius of the preceding curve was found to affect speed and speed differential. No significant relationship was found to exist between roadway geometry and speed on freeways.

Poe and Mason (2000) collected vehicle operating speed data along 27 urban collectors in Pennsylvania. Roadway, roadside, cross section, land use, and traffic engineering variables (speed limit, warning signs, and pavement markings) were collected for inclusion in an analysis to determine the urban characteristics that are associated with operating speed. Nu-Metrics Hi-Star NC-90 traffic counters were used to collect operating speeds along approach and departure tangents, as well as at the point of curve, middle of curve, and point of tangent. Field observation teams were used to track and document each vehicle passing through the study segment. The tracking was used to only include passenger cars, and to incorporate only free-flow vehicles, which consisted of vehicles with time headways of 5 s or more. Vehicles that were also impeded by bicyclists and pedestrians were excluded from the analysis. A mixed model approach was used to estimate the relationship between operating speed and roadway features. A mixed model includes both fixed and random effects in the specification. Fixed effects included roadway geometry and land use variables. Random effects are those that represent only a random sample of the population. The site location along the study segment was modeled as a random variable. A fixed-effects model was estimated for the midpoint of the curve. The model specification was

$$V = 57.47 - 0.228\ \text{DEGCVR} - 3.172\ \text{LANWIDN} - 1.229\ \text{HZRT5N}$$

where

V = mean speed (km/h);
DEGCVR = degree of curve (degrees);
LANWIDN = lane width (m); and
HZRT5N = roadside hazard rating.

Mixed models were also specified for the approach tangent, point of curve, midpoint of curve, and point of tangent. The resulting models were

PC150: $V = 49.59 + 0.5\ \text{DEGCVR} - 0.35\ \text{GRADE} + 0.74\ \text{LNWIDN} - 0.74\ \text{HZRT5N}$;
PC: $V = 51.13 - 0.10\ \text{DEGCVR} - 0.24\ \text{GRADE} - 0.01\ \text{LNWIDN} - 0.57\ \text{HZRT5N}$;
MID: $V = 48.82 - 0.14\ \text{DEGCVR} - 0.75\ \text{GRADE} - 0.12\ \text{LNWIDN} - 0.12\ \text{HZRT5N}$; and
PT: $V = 43.41 - 0.11\ \text{DEGCVR} - 0.12\ \text{GRADE} + 1.07\text{LNWIDN} + 0.30\ \text{HZRT5N}$.

It was concluded that the mixed modeling approach provided an appropriate approach to estimate speeds at multiple sites with multiple data collection points within each site. In the single point model for operating speeds at the midpoint of curve, the degree of curve, grade, lane width, and roadside hazard rating were associated with operating speed. As expected, speeds increase as lane width increases as vehicles enter and exit curves. However, data showed that increased lane width was associated with lower operating speeds within a horizontal curve for low-speed urban streets. This was attributed to wider lanes within curves, which allow truck access on roadways with tighter cross sections. When the analysis considered other data collection points within a site, it was concluded that the effects of site variability increase and the geometric effects decrease.

Tarris et al. (2000) argued that the design speed process can lead to inconsistent geometric designs. This is because there is no feedback loop in the design process to estimate operating speeds during the design process. Figure 1 illustrates the conceptual relationship of speed characteristics that was developed. This figure shows that the critical point design speed, or inferred design speed, can exceed the AASHTO design speed on low-speed urban streets, operating speeds exceed the AASHTO design speed, the critical point design speed varies along the alignment, operating speeds vary excessively along the roadway alignment, speed variance may be large in places, and the posted speed limit can exceed the AASHTO design speed. It was argued that the design process needs to produce a harmonious relationship between the target operating speed, the actual operating speed, and the posted speed limit. A conceptual process was recommended where

- The intended function of the roadway drives the selection of a target operating speed in the planning phase;
- The target operating speed is linked to the AASHTO-designated design speed in the geometric design phase, with a direct connection to driver expectancy and design consistency;
- The need and use of traffic control devices becomes complementary in the traffic engineering phase; and
- The actual operating speed is appropriate with the intended function of the roadway and the target operating speed during the vehicle operations phase.

Tarris et al. (1996) studied operating speeds on 27 urban collectors in Pennsylvania. The data collection and reduction procedures were the same as those described by Poe and Mason (2000). The analysis methods that were used in this study were OLS regression and a panel analysis approach. Aggregated and individual speed data were also modeled to show how aggregating speed data affects the model estimation and interpretation. This analysis was limited to the inclusion of the degree of curve as a predictor variable. Regression models were developed for individual and aggregate data.

For individual mean speed model: $V_{mean} = 53.8 - 0.272DC$ $R^2 = 0.63$

For aggregate mean speed: $V_{mean} = 53.5 - 0.265DC$ $R^2 = 0.82$

where

V_{mean} = mean operating speed (km/h) and

DC = degree of curve (degrees).

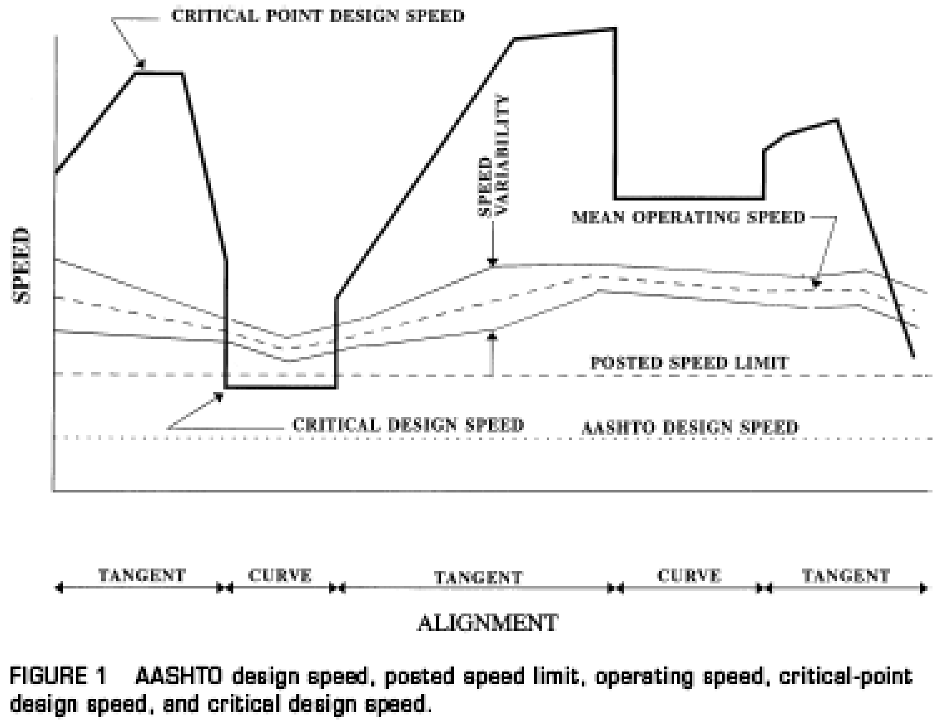

FIGURE 1 Conceptual speed relationships along a roadway alignment (Tarris et al., 2000).

As shown, the intercept and degree of curve parameter change nominally when changing from individual to aggregate-level data. The goodness of fit increases when using aggregate data to model mean operating speeds. The degree of curve was negatively correlated with the mean operating speed. Similar results were obtained when using a panel approach to model mean operating speeds. Based on the panel specification shown below, it is clear that including group effects in the model increased the goodness of fit. Including the time effects (i.e., sensor locations) resulted in little change to the model goodness of fit.

$V_{mean} = 52.18 - 0.231DC$	$R^2 = 0.70$	(individual drivers only)
	$R^2 = 0.49$	(degree of curve only)
	$R^2 = 0.79$	(degree of curve and group)
	$R^2 = 0.80$	(degree of curve, group, and time)

It was shown through this analysis that the degree of curvature is negatively correlated with operating speed on low-speed urban streets. The relationship was further explained through the use of panel analysis. It is recommended that future work on speed choice delve deeper into individual driver behavior.

Donnell et al. (2009) demonstrated the conceptual ideas in Figure 1 using actual field data collected along two-lane and multilane urban and rural roads of varying functional classifications. The paper explored speed harmony and speed discord, concepts linked to relationships between design speed, operating speed, and posted speed limit. Speed harmony was defined as a condition where operating speeds are consistent with the intended function of the highway or street and is therefore favorable with respect to safety and mobility. Speed discord was defined as a condition in which the design speed is lower than the posted speed limit, lower than various operating speed measures, or both. The authors found that the use of geometric design dimensions in excess of limiting values will result in observed speeds that exceed the designated design speed or regulatory speed limit. The authors concluded that relationships between design speed, operating speed, and posted speed limits should be important considerations in the geometric design of highways and streets. They also concluded that to produce speed harmony, it may be useful to include methods to predict operating or target speeds and to compute inferred design speeds during the design process.

Wang et al. (2006) also studied the effects of cross section characteristics and adjacent land use on operating speeds in Atlanta, Georgia. Speed data were collected using 200 vehicles equipped with GPS devices. A variety of drivers and vehicle types were included in the analysis. The vehicle types included passenger cars, minivans, SUVs, and pickup trucks. Tangent sections that were used as study corridors were long enough so that the longitudinal distance used by drivers to accelerate and decelerate near traffic control devices could be excluded from the sample. Corridors were also chosen that had as many different drivers as possible. This led to 35 candidate corridors. Analysis of speed profile plots showed that maximum tangent speeds were reached at different points by different drivers. Therefore, the 85th-percentile tangent speed along a corridor consisted of one observation for each driver traveling within the corridor. A mixed model approach was used to predict 85th and 95th percentile tangent speeds for urban streets. The models are as follows:

$$V85 = 31.565 + 6.491\ \text{lane.num} - 0.101\ \text{roadside} - 0.051\ \text{driveway} - 0.082\ \text{intersection} \\ + 3.01\ \text{curb} - 4.265\ \text{sidewalk} - 3.189\ \text{parking} + 3.312\ \text{land.use1} + 3.273\ \text{land.use2}$$

$$V95 = 31.143 + 6.671\ \text{lane.num} - 0.096\ \text{roadside} - 0.048\ \text{driveway} - 0.078\ \text{intersection} \\ + 3.324\ \text{curb} - 4.424\ \text{sidewalk} - 2.864\ \text{parking} + 3.507\ \text{land.use1} + 3.379\ \text{land.use2}$$

where

$V85$ = 85th percentile speed (mph);
$V95$ = 95th percentile speed (mph);
lane.num = number of lanes;
roadside = density of trees and utility poles (number/mile) divided by their average offset from roadway (ft);
driveway = density of driveways (driveways/mi);
intersection = density of T-intersections (intersections/mi);
curb = 0 if there is no curb; otherwise 1;
sidewalk = 0 if there is no sidewalk; otherwise 1;
parking = 0 if there is no on-street parking; otherwise 1;
land.use1 = 1 if land use is commercial; otherwise 0 (baseline is park and office land use); and
land.use2 = 1 if land use is residential; otherwise 0 (baseline is park and office land use).

It was found that roadside density, driveway density, intersection density, sidewalk presence, and parking presence were negatively associated with operating speed on urban streets. The number of lanes, presence of curb, and commercial and residential land uses were positively associated with operating speed. This research was not likely limited to free-flow vehicles only, since the GPS data did not have information related to the time headway between vehicles.

Gong and Stamatiadis (2008) studied rural four-lane highways in Kentucky. A total of 50 horizontal curves were used to develop separate OLS regression models of 85th-percentile speeds on inside (or left) and outside (or right) travel lanes. The model developed for the inside lane was

$$V_{85} = 51.520 + 1.567\ \mathrm{ST} - 2.795\ \mathrm{MT} - 4.001\ PT - 2.150\ \mathrm{AG} + 2.221 \ln(\mathrm{LC})$$

where

- V_{85} = the 85th-percentile speed of the inside or left lane (mph);
- ST = shoulder type indicator (1 if the shoulder type is surfaced; 0 otherwise);
- MT = median type indicator (1 if no median barrier present; 0 otherwise);
- PT = pavement type indicator (1 if pavement type is concrete; 0 if asphalt);
- AG = approaching section grade indicator (1 if the absolute grade ≥ 0.5 %; 0 otherwise); and
- LC = length of horizontal curve (ft).

The model explained nearly 65% of the variability in the 85th percentile inside lane operating speeds. In the model, a surfaced shoulder and logarithm of horizontal curve length were positively correlated with the 85th-percentile operating speed. The indicators for no median barrier, concrete pavement type, and the grade indicator variable were all negatively correlated with the 85th-percentile operating speed. The model developed by Gong and Stamatiadis (2008) for the outside (or right) lane on rural, multilane highways was:

$$V_{85} = 60.779 + 1.804\ \mathrm{ST} - 2.521\ \mathrm{MT} - 1.071\ \mathrm{AG} - 1.519\ \mathrm{FC} + 0.00047\ R + 2.408\ \mathrm{LC}/R$$

where

- FC = front curve indicator (1 if the approaching section is a curve; 0 otherwise) and
- R = horizontal curve radius (ft).

The model explained approximately 43% of the variability in the 85th-percentile outside lane operating speeds. In the model, a surfaced shoulder, horizontal curve radius, and the ratio of the horizontal curve length to radius were positively correlated with the 85th-percentile operating speed. The indicators for no median barrier, concrete pavement type, and the front curve indicator variable were all negatively correlated with the 85th-percentile operating speed. Fitzpatrick et al. (2001) investigated geometric, roadside, and traffic control device variables that may affect driver behavior on four-lane suburban arterials. Traffic signals and traffic volume were considered within the study site selection and data collection criteria and, therefore, were not included in the analysis. Regression models were estimated to determine how selected variables affect operating speed on horizontal curves and straight sections. When all variables were considered, posted speed limit was the most significant variable for both curves and tangent sections. Other significant variables for curve sections were deflection angle and access density class. In another series of analyses performed without using posted speed limit, only lane width was a statistically significant predictor variable for tangent sections, while median presence and roadside development were statistically significant predictor variables for curve sections. The analysis that included posted speed limit however, produced stronger relationships between speed and significant variables than the analysis that excluded posted speed limit. Table 19 lists the findings from the regression analysis.

The horizontal curve data from the above study were also used to identify the location of the minimum speed within the curve (Fitzpatrick et al., 2000a). A total of 23 horizontal curves were included in the evaluation. Because the accuracy of the laser guns was 1.6 km/h (1 mph), speed profiles for each site were searched for the absolute minimum speed, then any speed within 1.6 km/h (1 mph) within that speed was identified. Table 20 illustrates the results using 5% increments of the curve lengths. A visual inspection shows the range where speeds are most frequently at a minimum value spans the mid and three-quarter points of the curves. This finding is fairly consistent throughout the range of radii shown (although for the larger radius curves, the amount of minimum speed found in the curve decreases).

Himes and Donnell (2010) investigated the effects of roadway geometric design features and traffic flow on operating speed characteristics along rural and urban four-lane highways in Pennsylvania and North Carolina. A simultaneous equations framework was used to model the speed distribution, developing equations for the mean speed and standard deviation of speed for both travel lanes using the three-stage least squares estimator. This simultaneous equation modeling framework was first introduced by Shankar and Mannering (1998) to model speeds on

TABLE 19 Findings from Regression Analysis by Fitzpatrick et al. (2001)

Variables	***Parameter Estimate***	***p*-value**
Final Analysis with Speed Limit for Horizontal Curve Sites R^2 = 75%; F-statistic = 15.341; adjusted R^2 = 71%		
Intercept	43	0.0001
Speed limit (km/h)	0.52	0.0001
Deflection angle (deg)	–0.15	0.0183
Access density [if below 12 pts/km (19.3 pts/mi) then 1, otherwise 0]	4.4	0.0262
Final Analysis Without Speed Limit for Horizontal Curve Sites R^2 = 62%; F-statistic = 15.265; adjusted R^2 = 52%		
Intercept	45	0.0001
Median Presence (if Raised or TWLTL then 1, otherwise 0)	9.2	0.0023
Roadside: if school then 1, otherwise 0	13	0.0733
Roadside: if residential then 1, otherwise 0	18	0.0032
Roadside: if commercial then 1, otherwise 0	19	0.0025
Final Analysis with Speed Limit for Straight Section Sites R^2 = 54%; F-statistic = 40.503; adjusted R^2 = 53%		
Intercept	29	0.0002
Speed limit (km/h)	0.70	0.0001
Final Analysis Without Speed Limit for Straight Section Sites R^2 = 27%; F-statistic = 12.594; adjusted R^2 = 25%		
Intercept	19	0.2345
Average lane width (m)	15	0.0012

TABLE 20 Summary of Minimum Speed Ranges (Fitzpatrick et al., 2000a)

a freeway segment in Washington State. It was later explored in depth and compared to limited information (e.g., OLS regression) and full-information (e.g., seemingly unrelated regression) modeling methods by Porter (2007).

Himes and Donnell (2010) found that different geometric design features were associated with mean speed and speed deviation in the left- and right-lanes. The heavy vehicle percentage in the right lane was the only traffic flow characteristic that was significantly associated with the speed distribution. Other nongeometric features were associated with the speed distribution, including the posted speed limit, intersection signalization, and the adjacent land use. The authors concluded that future multilane highway speed models consider using a simultaneous equations framework to account for the endogenous relationship between lane mean speeds and speed deviation. The framework developed as part of the research allowed for consideration of the entire speed distribution.

As a summary and based on a review of the existing literature, several geometric design variables appear to be associated with operating speeds along rural highways and urban streets in North America. Although the objectives and scope of the studies have varied, the following relationships appear to converge for studies reported along low-speed urban streets.

- The presence of median is associated with higher operating speeds,
- Operating speeds decrease as the degree of horizontal curve increases,
- Operating speeds decrease as the roadside becomes more restricted (i.e., objects closer to roadway),
- Operating speeds decrease as the vertical grade increases,
- Operating speeds decrease as the access density increases, and
- The presence of parking and sidewalks are associated with lower operating speeds.

Few research studies have investigated the relationship between several geometric variables and operating speeds along rural multilane highways. Based on the existing literature, the following relationships have been reported.

- Free-flow operating speeds decrease as the access density increases,
- The presence of a paved shoulder is associated with higher operating speeds,
- Increasing the vertical grade is associated with lower operating speeds, and
- Increasing the length or radius of horizontal curve is associated with higher operating speeds.

Several research studies have investigated the relationship between geometric variables and operating speeds along suburban highways. Based on the existing literature, the following relationships have been reported.

- Operating speeds increase as the available sight distance increases,
- Operating speeds decrease as the access density increases,
- Operating speeds decrease as the roadside becomes more restricted (i.e., objects closer to roadway),
- The presence of a paved shoulder is associated with higher operating speeds,
- Increasing the radius of horizontal curve is associated with higher operating speeds,
- Increasing the deflection angle of the horizontal curve is associated with lower operating speeds, and
- Increasing the curvature change rate or ratio of successive horizontal curves is associated with lower operating speeds.

CHAPTER 3

Speed Models in Europe

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GERMANY

Operating Speed

The operating speed in Germany is officially defined as the 85th-percentile free-flow speed. The operating speed was first used in the design of two-lane rural roads in the 1973 edition of the *German Guide for the Design of Highways* RAL-L-1 (1973). It was also the first time that the operating speed became a design criterion for rural two-lane highways in an endeavor to address real-world parameters in the design process and to relate this pragmatic road user speed with other speed concepts such as design speed or speed limit. Since then, this definition of the operating speed played a significant role in all subsequent editions of the German Design Guide.

Historical Development

Dilling Model

The first operating speed model reported in Germany based on field measurements goes back to Dilling (1973).

$$V_{85} = 5.32 + 1.12 \times V_{\text{avg}}$$
$$V_{\text{avg}} = 25.10 + 5.57 \times b + 0{,}05 \times R - 0.05 \times \text{CCR}$$

where

V_{avg} = average curve speed (km/h),
b = lane width (m),
R = curve radius (m), and
CCR = curvature change rate of the highway section (gon/km).

This model relates the operating speed (V_{85}) with the average curve speed V_{avg}, the lane width b, and the curvature change rate. Since the derivation of the above formula was based on a limited number of observations, it was not useful for practical purposes.

Lamm Model

Lamm (1973) published operating speed models as they resulted from 200 measurements on highways with a vertical grade between –4% and +2%. These models are

New highways: $V_{85} = 84.19 + 37.80 \times e^{-\mathrm{SP}/152}$ pavement widths B ≥ 8.50 m
Old highways: $V_{85} = 69.75 + 36.80 \times e^{-\mathrm{SP}/201}$ 6.00 m ≤ B ≤ 7.50 m

In the above equations, the highway section specific parameter SP was introduced as

$$\mathrm{SP} = \frac{\sum_{i=1}^{i=n} \frac{|L_i|}{|R_i|} \times 63.7 \times k_i + \sum_{i=1}^{i=n} \frac{|L_i|}{|2R_i|} \times 63.7 \times k_j}{S_1 + S_2}$$

where

SP = highway section specific parameter (gon/km);
$L_{i/j}$ = circular arc length i or transition curve length j (m);
$K_{i/j}$ = curvature factor = V_0 / $V_{i/j}$;
V_0 = theoretical speed for circular arc with a radius of 500 m;
$V_{i/j}$ = theoretical speed for radii R_i < 500 m, and for transition curves $R_j = 2 \times R_i$;
S_1 = sight distance up to the reference cross-section; and
S_2 = sight distance from the reference cross-section.

The principal idea behind these models is that the operating speed ceases to increase beyond horizontal curve radii values of 500 m. Since this model was based on cross section measurements, it can only be of value for dense cross section speed analysis and does not provide an operating speed overview for the whole curved site.

Trapp and Oellers Model

Trapp and Oellers (1974) managed to express the instantaneous speed of a passenger car as a function of various alignment design parameters according to the following formula:

$$V_s = 11.00 + \sum \Delta V_i$$

where

V_s = Instantaneous speed of a highway section (km/h) and
ΔV_i = impact of lane width; climbing lane width; longitudinal slope, sight distance; curvature change rate.

This model could not be considered because of possible auto-correlation between the influencing parameters.

Koeppel and Bock Model

Koeppel and Bock (1979) provided two relationships for the average instantaneous speed as follows.

Model 1:

$$V_{LEv} = 54.48 - 0.0426 \times \mathrm{CCR} + 4.361 \times b + 0.0271 \times \mathrm{SD}$$

Model 2:

$$V_{\mathrm{LE}v} = 57.49 - 0.0596 \times \mathrm{CCR} + 0.0000258 \times (\mathrm{CCR})^2 + 4.293 \times b + 0.0222 \times \mathrm{SD}$$

where

$V_{\mathrm{LE}v}$ = average instantaneous speed over the whole highway section (km/h);
CCR = Curvature change rate of the influencing highway length given from developed empirically diagrams;
b = Pavement width including edge strip width (m); and
SD = average sight distance within 400 m ahead of the referred cross section.

Al-Kassar et al. Model

Al-Kassar et al. (1981) succeeded in defining an operating speed model through car following along 72 km of rural roads in Germany. They estimated the following model (R^2 = 0.78):

$$V_{85} = 440.74276 - 15.17804 \times 0.7^{\mathrm{CCR}} - 2.74417 \times \mathrm{CCR}^{0.5} - 0.35627 \times G^2 + 37.14797 \times b - 218.51481 \times b^{0.5} - 41.82062 \times \mathrm{ES} + 38.23629 \times \mathrm{ES}^{0.5} + 65.16351 \times \mathrm{PS} - 70.84485 \times \mathrm{PS}^{0.5}$$

where

V_{85} = 85th percentile speed (km/h);
CCR = curvature change rate along 900 m of the road section (500 m ahead and 400 m backward of the speed measurement;
G = longitudinal slope;
b = pavement width along 100 m ahead and 200 m backward of the speed measurement (m);
ES = edge strip width 100 m ahead and 200 m backward of the speed measurement (m); and
PS = paved shoulder width (m).

The application of the above model was difficult to apply in practice for the following reasons: (*a*) difficulty of measuring the sliding mean value of the parameters along a specified road segment; (*b*) its application assumes consistent alignments that limits the effectiveness of the model; and (*c*) the influential parameters for determining the driver's speed choice should better rely on parameters the driver anticipates as such and not by just values of design parameters the driver cannot interpret.

Durth et al. Model

Durth et al. (1983) estimated the following operating speed model using data from Germany.

$$V_{85/R} = 3.039 + 1.3035 \times V_{85/\mathrm{total}} - \frac{2059}{R} + \frac{42394}{R^2}$$

where

$V_{85/R}$ = 85th-percentile speed of a curve (km/h);
$V_{85/\mathrm{total}}$ =85th-percentile speed of the whole road section (km/h); and
R = curve radius (m) and $R \geq 62$ m.

Although the model considers the driver's operating speed choice it can only be applied in cases where there is a smooth variation of the operating speed along a road section. If there are strong variations of the operating speed value along the road section this model fails to predict accurately the speed choice of the driving population.

Biedermann Model

A model was formulated by Biedermann (1984):

$$V_{85} = 62.17 + 10.11 \times b - 189 \left(\frac{b}{R_m}\right) - 593 \times \left(\frac{b}{R_m}\right)^2$$

where

V_{85} = 85th-percentile speed (km/h);
R_m = mean radius along a road section 30 m ahead and 60 m backwards of the speed measurement position (m); and
b = pavement width (m).

As with the previous model, one main disadvantage of this model is its strong correlation with a smooth and consistent highway alignment, failing again to accurately predict the real value of the driver's speed choice.

Koeppel Model

Building on a previous effort to predict the operating speed on two-lane highways by Koeppel and Bock (1979), Koeppel (1984) carried further measurements that led to the formulation of the following operating speed model:

$$V_{85} = 0.065 + 0.484 \times V_{50} + 1.869 \times {V_{50}}^2 \times 10^{-2} - 1.349 \times {V_{50}}^3 \times 10^{-4}$$
$$V_{50} = 65.23 + 4.293 \times b - 0.0756 \times \mathrm{CCR} + 0.0000364 \times \mathrm{CCR}^2$$

where

V_{50} = 50th-percentile speed (km/h),
b = pavement width (m), and
CCR = curvature change rate (gon/km).

The above model however did not bring any significant improvement in the prediction value of the model because of the strong correlation between sight distance and CCR values.

Buck Model

After defining various types of alignments with varying combinations of CCR values and gradients, Buck (1992) estimated the following model:

$$V_{85} = 125.3 - \frac{520}{X_1} - \frac{2110}{X_2} - 7.20 \times X_3 - 32.5 \times X_4 - 0.59 \times X_5 - 0.23 \times {X_5}^2$$

where

X_1 = local sight distance (m),
X_2 = mean stopping sight distance 400 m backwards (m),
X_3 = local curvature $100/R$ (1/m),
X_4 = mean curvature 400 m backwards 100/R (1/m), and
X_5 = mean longitudinal slope 400 m backwards (%).

Although the model considered a wide variety of alignments, it is highly influenced by the selected radii, sight distances and pavement widths of the alignment, which cannot be adapted without further investigation on other types of alignments.

Lippold Model

Lippold (1997) estimated models for individual curves, in contrast to the German Guide for the design of highways that used mean operating speed values along road sections with similar alignment characteristics. The following models for individual curve radii R were reported (all units in metric).

- $V_{85} = -4.880 + 18.2222 \times \ln(R)$ as a universal function for pavement widths greater than or equal to 6 m wide.
- $V_{85} = 7.8066 + 16.0274 \times \ln(R)$ for pavement widths greater than or equal to 6 m and less than 7 m wide.
- $V_{85} = -2.8981 + 17.8093 \times \ln(R)$ for pavement widths greater than or equal to 6 m and less than 7 m wide and radii less than or equal to 200 m.
- $V_{85} = -24.379 + 22.0465 \times \ln(R)$ for pavement widths greater than or equal to 7 m wide.

Bakaba Model

Bakaba (2003) developed a model for the operating speed in Germany. The model incorporates a new approach by incorporating the undulation of the vertical alignment (vertical curvature change rate) as an independent parameter. This parameter is defined as

$$\mathrm{WE} = \left(\frac{\sum_{i=1}^{n} \left| \arctan\left(\frac{G_{i1}}{100}\right) - \arctan\left(\frac{G_{i2}}{100}\right) \right|}{L} \times \frac{200}{\pi} \right)$$

where

WE = vertical alignment undulation on a specified road section (gon/km),
G_{i1} = slope of the first tangent of the vertical curve (at point TC) (%),
G_{i2} = slope of the second tangent of the vertical curve (at point CT) (%),
N= number of vertical curves along the road section (–), and
L = length of the road section (km).

The model also relies on a so-called reference speed that takes into account the roadway alignment characteristics (Figure 2), where from by applying some correction values depending on the sequential differences in CCR values the acceleration and deceleration rates of the driving task are determined along a determined length. Thus a speed profile is constructed that reflects the driver's speed choice along the roadway with the specific characteristics. Due to many sequential calculations needed for each step of the speed profile, a corresponding calculation subroutine is needed to handle many calculations that are necessary for the speed profile construction. In general, the model has not yet been tested for its practical relevance in the safety evaluation of two-lane rural highways.

Official Approach

The concept of operating speed has been officially in use by the German Guide for the Geometric Design of Highways for about 35 years. Its first documentation as an evaluation parameter for alignment consistency is found in the edition of German Guide (RAL-L-1, 1973). The same evaluation process with relation to the operating speed was kept practically unchanged in the next edition of German Guide (RAS-L-1, 1984). The basis for determining the operating speed for both Guides is shown in Figure 3. The diagram provided the mean operating speed (85th-percentile speed) along sections with similar geometric characteristics expressed through similar CCR values (deflection angle change rates of the horizontal alignment).

The next edition of the German Guide for the Design of Highways (RAS-L, 1995) provided a two-way operating speed determination for new and existing roads. New alignments are divided into sections with relatively consistent horizontal alignment characteristics (characterized by CCR). The operating speed value is determined according to Figure 4 as a function of the CCR value of the whole road section and represents an average 85th-percentile speed value along a section of roadway (applicable to either travel direction). The operating speed values for each section are compared; the comparison is used as a basis for determining inconsistencies in the alignment between successive sections.

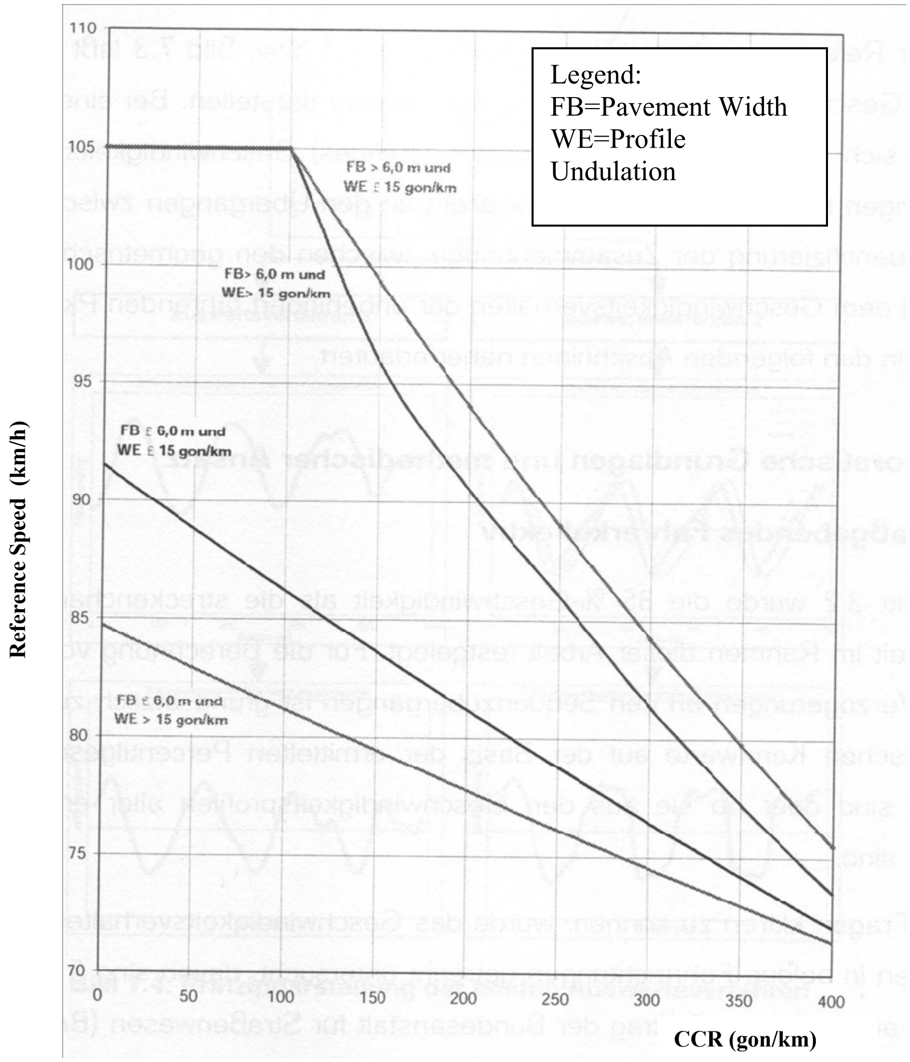
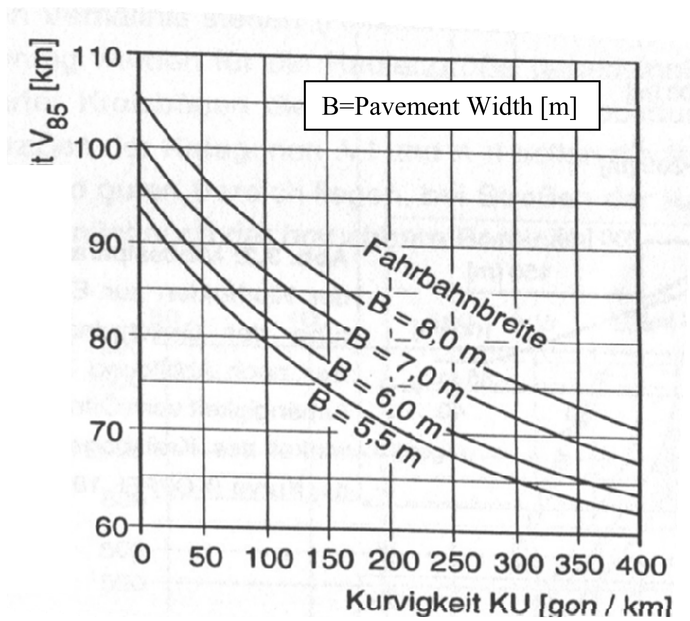


FIGURE 2 Definition of reference speed for determining the operating speed profile according to Bakaba (2003).

For older, existing horizontal alignments, the German Guide of 1995 provided the diagram of Figure 5 for the determination of the operating speed as a function of the individual curve radius *R*.

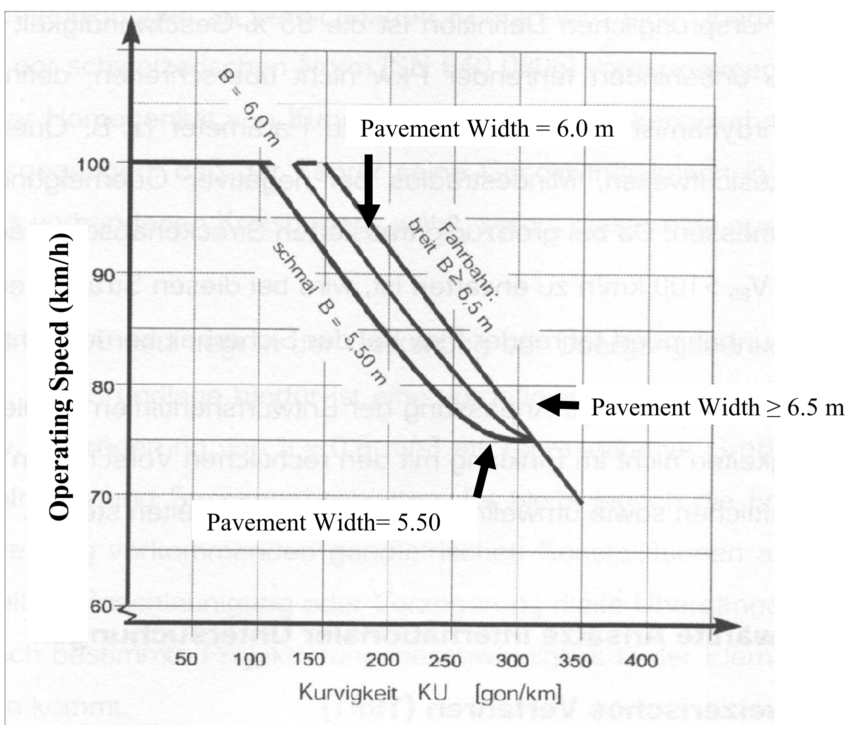
Further Development

In the new German Guide for the Design of Rural Roads expected to be released at the end of 2010, a new road design approach is introduced. This new design approach makes use of design classes of highways without any reference to the operating speed, making the whole issue of operating speed obsolete for consideration in future road design. No further details can be provided at this point until the new Guide is released.



CCR

FIGURE 3 Operating speed determination background according to the corresponding German Design Guides of 1973 and 1984.



CCR (gon/km)

FIGURE 4 Operating speed determination for consistent horizontal alignments according to the German Guide of 1995.

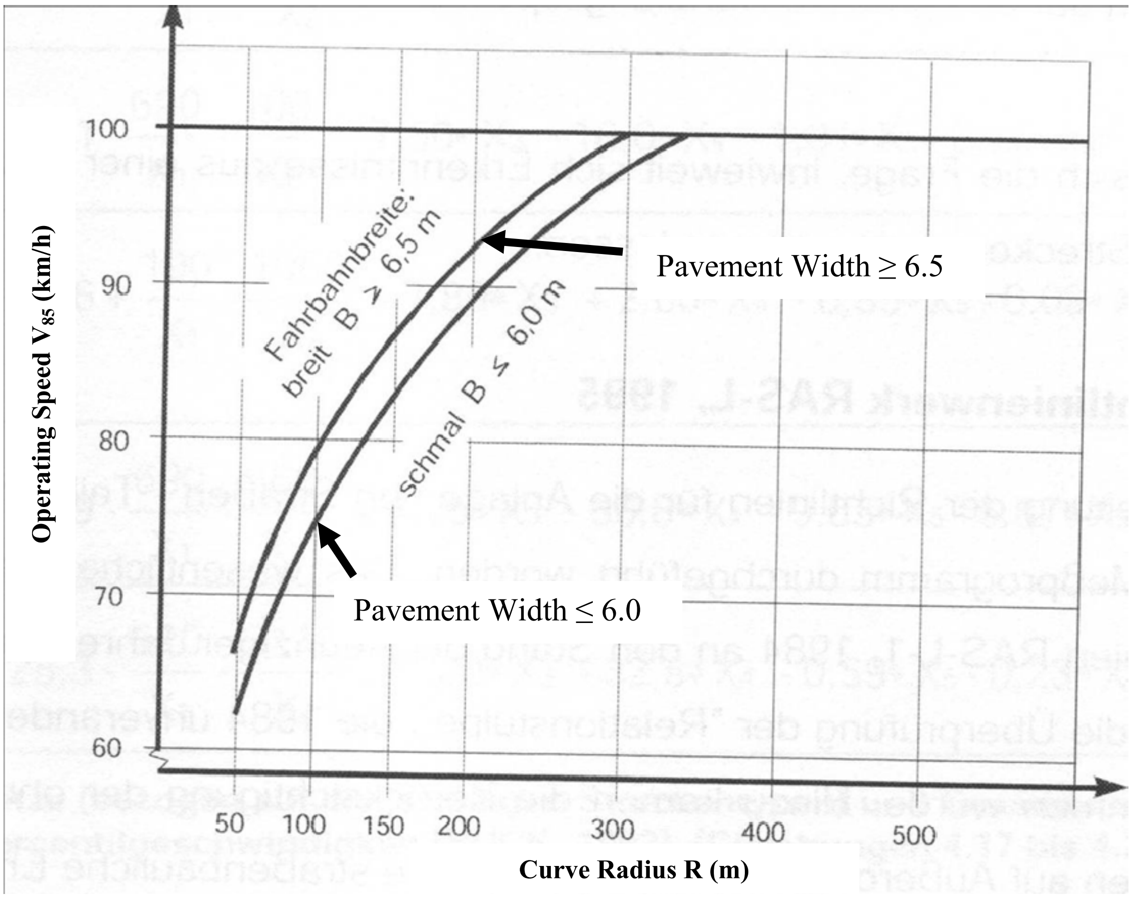


FIGURE 5 Operating speed determination for existing horizontal alignments according to the German Guide of 1995.

SWITZERLAND

Definition

Swiss Guide SN 640 080b of 1991 (VSS, 1991) provides an extensive and detailed description about the determination of the individual operating speed, called Project Speed V_P, as well as the derivation of its profile along the route of a highway. According to this guide, the operating speed is the highest speed with which a specific section of a highway can be traversed safely. An appropriate profile of the operating speed warrants a homogeneous alignment and consequently the traffic safety of the roadway alignment. It forms the basis for the determination of the necessary sight distances, the minimum radii of the vertical curves and the superelevation rates. The basic assumptions for its development are

- Drivers select the speed to negotiate a curve based on the horizontal curve radius as far as the curve is long enough to be recognized as a curve;
- Influence of the gradient can be neglected for grades less than 7%;

- Vehicles are regarded as point masses which negotiate a curve following an ideal lane track;
- Friction values correspond to the sliding friction coefficients of a profiled tire on a clean and wet pavement—the coefficient of friction depends on the speed of the vehicle;
- The sliding friction coefficient μ is composed by a tangential friction component fL and a radial one fR for which approximately the following relationship holds true: $\mu = \sqrt(f_L^2 + f_R^2)$;
- For purposes of short stopping distances on curves 90% of the available friction coefficient is assigned to the tangential friction component;
- Operating speeds V_P (km/h) corresponding to a curve radius *R* (m) are derived from equation;
- $V_P = \sqrt{127 * R * (f_R + e)}$, where *e* = the superelevation rate of the curve = 0.07; and
- Operating speeds for various curve radii are obtained from Table 21.

For curve radii that lie between the values in Table 21, the next higher speed is always selected, i.e., for *R* = 100 for example, V_P = 60 km/h is selected. On tangents the highest speed limit allowed for the specific road category is selected. This also applies to curve radii that allow higher speeds than the existing legal speed limit for the particular road category.

Speed Profile

For the drawing of the operating speed profile according to the Swiss Guide (VSS, 1991) the following assumptions are made.

1. Drivers select operating speeds in accordance with the curve radii taking into account the existing speed limits.
2. Operating speeds remain constant along the entire length of the curve.
3. Drivers select their operating speed in accordance with the next following curve (circular arc) even though the second next curve lies in the sight view of the driver.
4. On tangents and transition curves operating speeds correspond to the existing speed limits.
5. Deceleration ends at the beginning of the circular arc.

TABLE 21 Operating Speeds as a Function of the Curve Radii in Switzerland

Radius *R* (m)	45	60	75	95	120	145	175	205	240	280	320	370	420	470	525	580	≥ 650
V_P (km/h)	40	45	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120

6. Acceleration starts at the end of the circular arc.
7. Acceleration and deceleration rates are set equal to a=0.8 m*sec^{-2}.
8. The project distance D_T corresponds to the road section where the operating speed between two design elements changes from the value V_{P1} to the value V_{P2} as follows:

$$D_T = \frac{\Delta V \times V_m}{12.96 \times a}$$

where

D_T = acceleration or deceleration length (m);
ΔV = speed difference $|V_{P1} - V_{P2}|$ (km/h);
V_m = mean speed between two successive road sections: $V_m = \frac{V_{P1} + V_{P2}}{2}$; and
a = acceleration or deceleration rate = 0.8 m * sec^{-2}.

As a decision distance D_E it is understood the distance a driver needs to carry out maneuvers to overcome obstacles. It is empirically calculated according to the formula:

$$D_E = 12 \times V_P$$

where V_p is expressed in m * sec^{-1}.

Deceleration distances are always less than the decision distance D_E. Table 22 provides the decision distances D_E as a function of the successive speeds V_{P1} and V_{P2}.

Creation of Speed Profile

The way the speed profile is created according to the Swiss Guide is illustrated in Figure 6. When creating speed profiles, various cases may appear due to various possible combinations of design elements. The most frequent cases are shown in Figure 7. Cases 6 and 7 are exceptional cases and are generally to be avoided for new highways. They can be implemented on existing roads mainly by taking supplementary safety measures.

Implementation of the Speed Profile

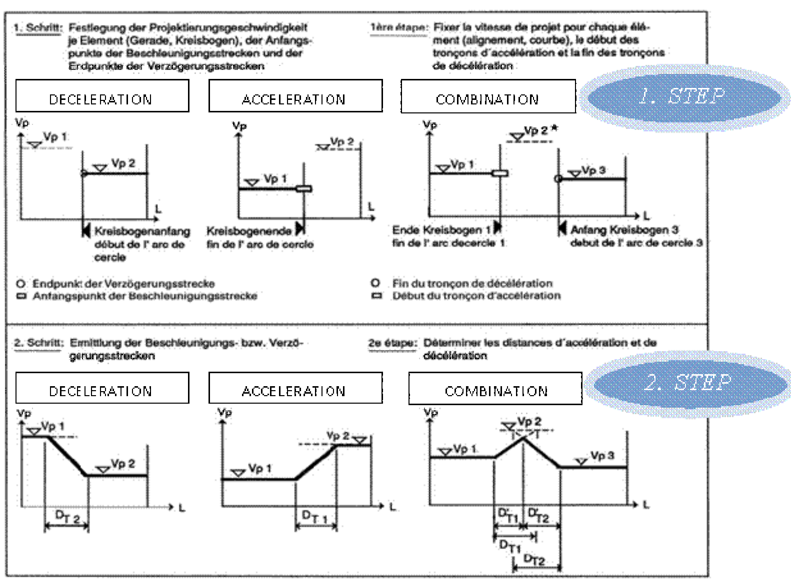
According to the Swiss Guide, it is important from a safety point of view to control the deceleration of vehicles. Therefore the speed profile for both directions of travel is created and examined. The following rules apply.

TABLE 22 Values of the Decision Distance d_e for the Determination of the Speed Profile on Swiss Roads

V_{P1} resp. V_{P2} [km/h] \ V_{P2} resp. V_{P1} [km/h]	40	45	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120
40																	
45	20																
50	45	25															
55	70	50	25														
60	95	75	55	30													
65	125	105	85	60	30												
70	160	140	115	90	65	35											
75	195	175	150	125	100	70	35										
80	230	210	190	165	135	105	70	35									
85	270	250	230	205	175	145	110	75	40								
90		295	270	245	220	185	155	120	80	40							
95			315	290	260	230	200	165	125	85	45						
100					310	280	245	210	175	135	90	45					
105						330	295	260	225	185	140	95	50				
110							345	310	275	235	195	145	100	50			
115								365	330	290	245	205	155	105	55		
120									385	345	305	260	210	165	110	55	

$D_T > D_E$

1. $D=D_T$ and $V_P < V_{Pmax}$ in a balanced ratio and the length of the Transition Curve or Tangent is optimal



Legend:

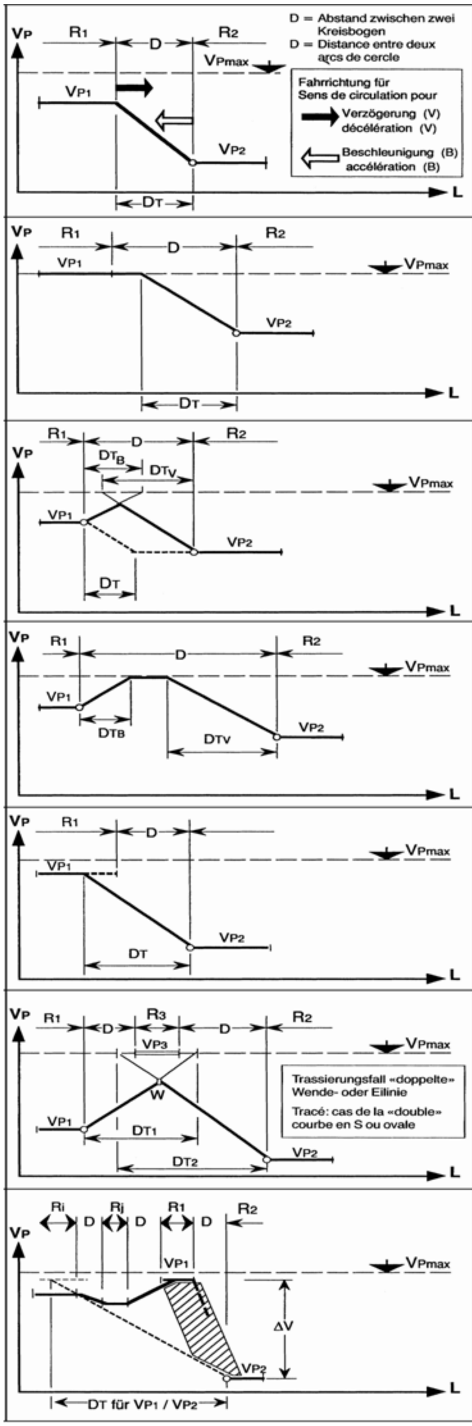
D′$_{T1}$, D′$_{T2}$ Transition sections for acceleration/deceleration due to mutual influence between two design elements

***:** In the case of reversed transition curves the operating speed of the tangent is applied

O: End of the deceleration section

☐**:** Beginning of the acceleration section

FIGURE 6 Creation of the speed profile according to the Swiss Guide.



2. *$D=D_T$ and $V_P < V_{Pmax}$* *It is applied when Radii R_1 and R_2 are in a balanced ratio and the length of the transition curve or tangent is optimal.*
3. *$D>D_T$ and $V_{P1} = V_{Pmax}$* *It is applied when the greater radius corresponds to $V_P = V_{Pmax}$ usually in the sequence tangent–circular arc.*
4. *$D>D_T$ and $V_P < V_{Pmax}$* *This is the case when the length of the transition curves and tangent within is greater than the Distance D_T needed to decelerate from V_{P1} to V_{P2}.*
5. *$D>>D_T$ and $V_P < V_{Pmax}$* *In this case the maximum Speed V_{Pmax} is attained between two circular arcs due to long tangents in between ($D>D_{TB}+D_{TV}$).*
6. *$D<D_T$* *It is applied when the distance between two circular arcs is too short; due to too short lengths of transition curves and tangent.*
7. *$D<D_T$ between two circular arcs* *It is applied when Radii R_1 and R_2 are in a unbalanced ratio (R1, R2 << R_3) and the length of the intermediate curve and distances (D) are too short.*
8. *$D<<D_T$ and ΔV is big (extreme case)* *It is applied when difference between V_{P1} and V_{P2} (ΔV) is so big that deceleration needed between curves R_1 and R_2. Usually when design elements are too short.*

FIGURE 7 Most important cases of the speed profile according to the Swiss Guide.

- For high-performance highways, i.e., highways with a design speed varying between 80 and 120 km/h:
  - Moving from a highway section with a very gentle alignment to another highway section with an operating speed less than 120 km/h, the operating speed differences should lie under 10 km/h. A gentle alignment is defined as a sequence of tangents and curves with radii equal to or greater than 3,000 m.
  - Between two successive curves speed differences should not be more than 20 km/h and if possible less than 15 km/h.
  - The existing sight distance ahead of a curve should be greater than the transition length D_T.
- For major highways, i.e., highways with a design speed varying between 60 and 80 km/h, as well as connecting highways, i.e., highways with a design speed between 50 and 80 km/h:
  - Moving from highway section with a gentle alignment to another one with an operating speed less than 80 km/h, operating speed differences should be under 5 km/h. As a gentle alignment it is comprised the one with tangents and curves that have radii values no less than 420 m and whose total length *D* according to Figure CH-2 including the transition curves length is greater than the decision distance DE.
  - Between two successive curves speed differences should lie under 20 km/h and if possible less than 10 km/h.
  - The existing sight distance ahead of a curve should be greater than the transition length DT.
  - On existing highway sections where the above rules cannot always be applied the creation of the corresponding speed profile leads to the identification of unsafe highway locations where special treatments are needed. In cases where the application of the above mentioned rules lead to various conflicts, it should be examined whether it is possible.
- Reduce the operating speed differences between successive curves through reducing the speed radii differences.
- Keep the minimum lengths of tangents, transition curves and circular arcs according to the Guide SN 640100 (Elements of the Horizontal Alignment).
- Improve the sight distance conditions.

If the conflicts cannot be resolved then special signing and marking of the highway section is needed.

Development

New operating speed measurements were carried out in Switzerland in the year 1998 on rural two-lane roads with a general speed limit of 80 km/h. The curve radii varied between 18 and 700 m. The following findings were reported.

- The speed in the circular arcs appeared to be more homogeneous and was reduced than those of the year 1978 when the speed limit was 100 km/h (see Figure 8). Consequently radial acceleration rates and radial friction demands have also been reduced.

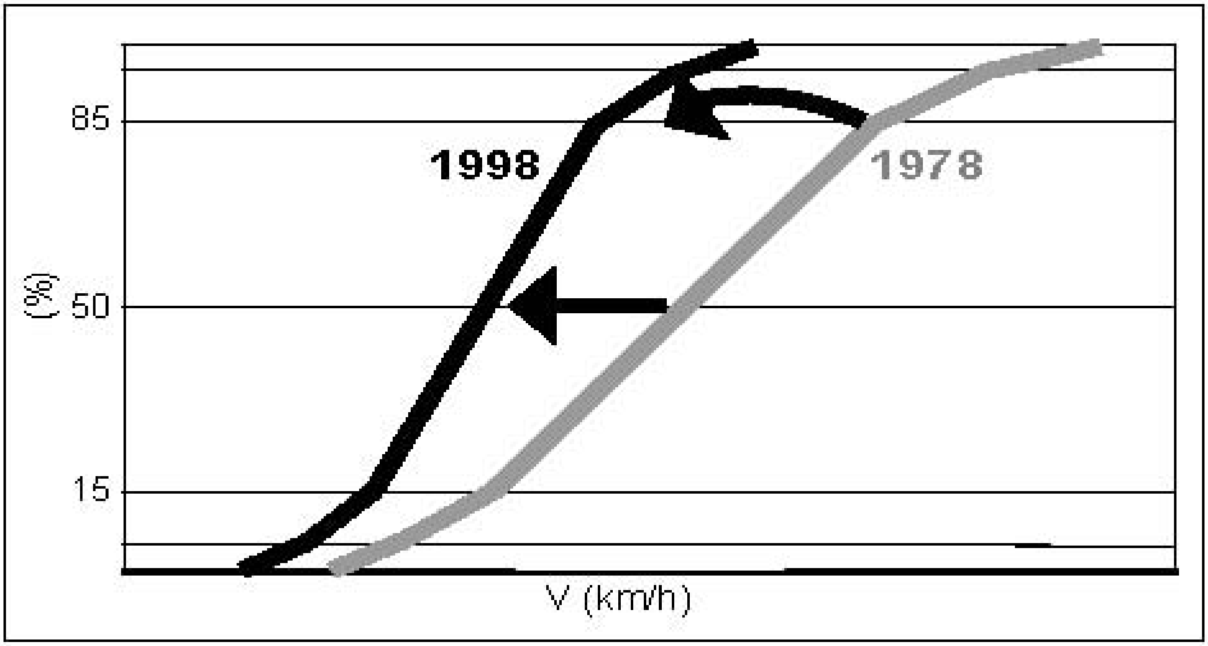
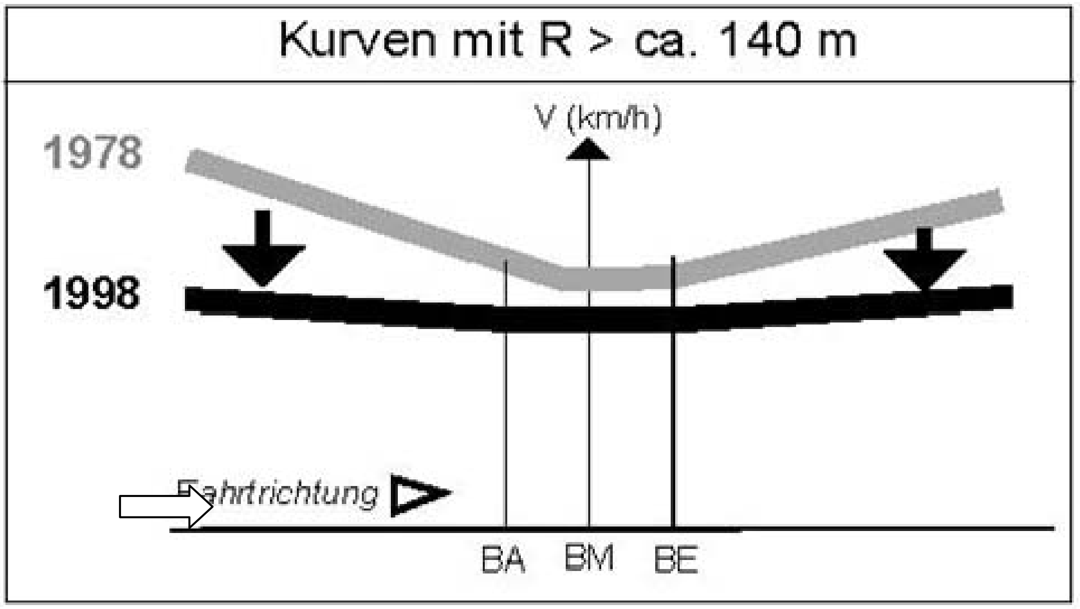


FIGURE 8 Speed distribution along curves in Switzerland in the years 1978 and 1998.

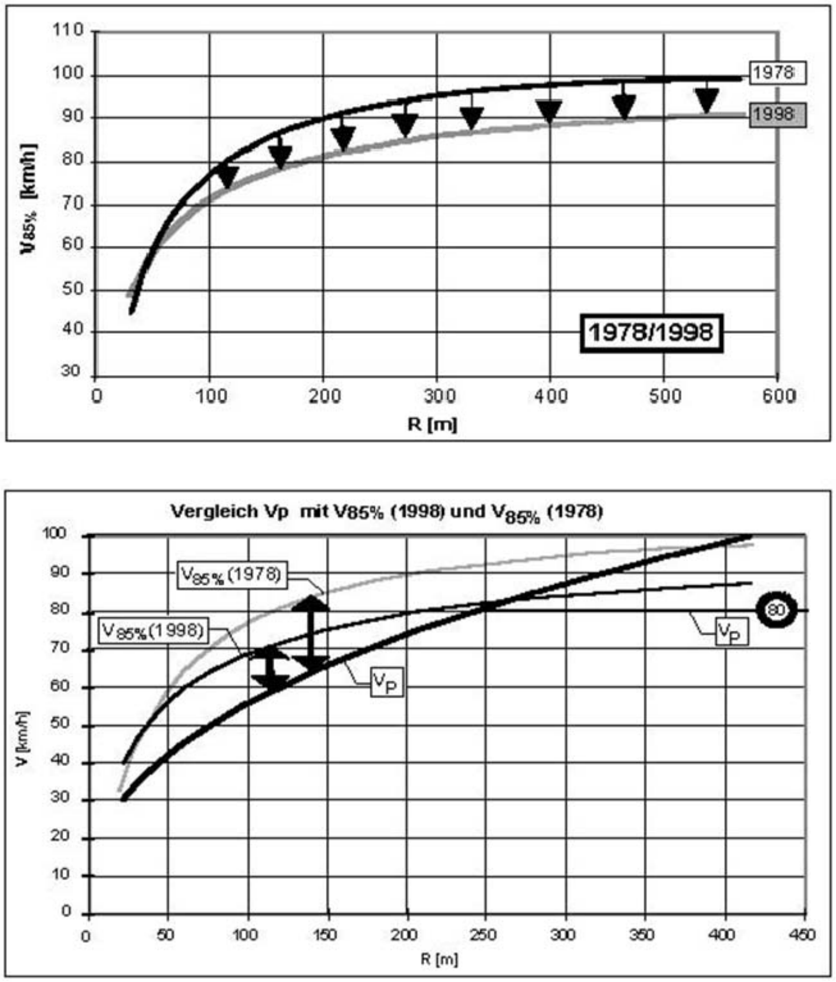
- The speed differences $\Delta V85\%$ and $\Delta V15\%$ as well the standard deviations have been reduced within the 20-year period.
- The U-form of the speed profile within the circular arc (deceleration ahead of the arc/acceleration after the arc) has been practically vanished for curve radii greater than 140 m (see Figure 9).
- For sharp curves (radii less than 115 m) the deceleration rates correspond fully to the speed profile model assumed in the Swiss Guide.
- A correlation between curve radius and operating speed has not been found to be significant. As correlated geometric parameters have been discovered to be the circular arc length and the deflection arc (Figure 10). Some influence impose also the lane width and the sight distance.
- The introduction of the new speed limit has led to a reduction of the operating speed by an amount of 10 km/h in general. However, for curve radii above 150 m the 85th-percentile speeds lie above the speed limit of 80 km/h (see Figure 11).
- Small deflection angles (less than 50 gon) lead to increased speeds. Therefore circular arcs that allow a time of less than 2 s to accommodate seem to be inappropriate for design.



Direction of Travel

FIGURE 9 Typical speed profile along a curve with radii greater than 140 m.

FIGURE 10 Curve parameters found to significantly influence operating speed in curves.



V_P: Operating Speed according to Swiss Guide (1)

FIGURE 11 Development of operating speeds on curves in the 20-year period 1978–1998, in Switzerland as correlated to the speed limits and to the speed modeling of the Swiss Guide.

According to another research project related to the development of speeds of passenger cars and trucks on upgrade sections, it could be demonstrated that grades up to 8% do not influence the operating speed of passenger cars. This value therefore is proposed to substitute the 7% limiting grade mentioned in the official document in its next edition.

ITALY

Official Approach

In Italy, the official speed profile modeling used by the Italian Ministry of Infrastructure and Transport is based on the project speed and accepted acceleration–deceleration rates of 0.8 m/s^2

ignoring the influence of any other parameter. Project speed according to the Italian Standard for the geometric design of roads, IT-1, is the speed derived by the known formula of vehicle dynamics on curves.

$$e + f = \frac{V_P^2}{127 \times R}$$

where

V_P = project speed (km/h);
e = superelevation rate;
f = friction values ranging from 0.22 at 25 km/h to 0.09 at 140 km/h; and
R = curve radius (m).

For the transition length between two speed values the following formula is used:

$$D_T = \frac{\Delta V \times V_m}{12{,}96 \times a}$$

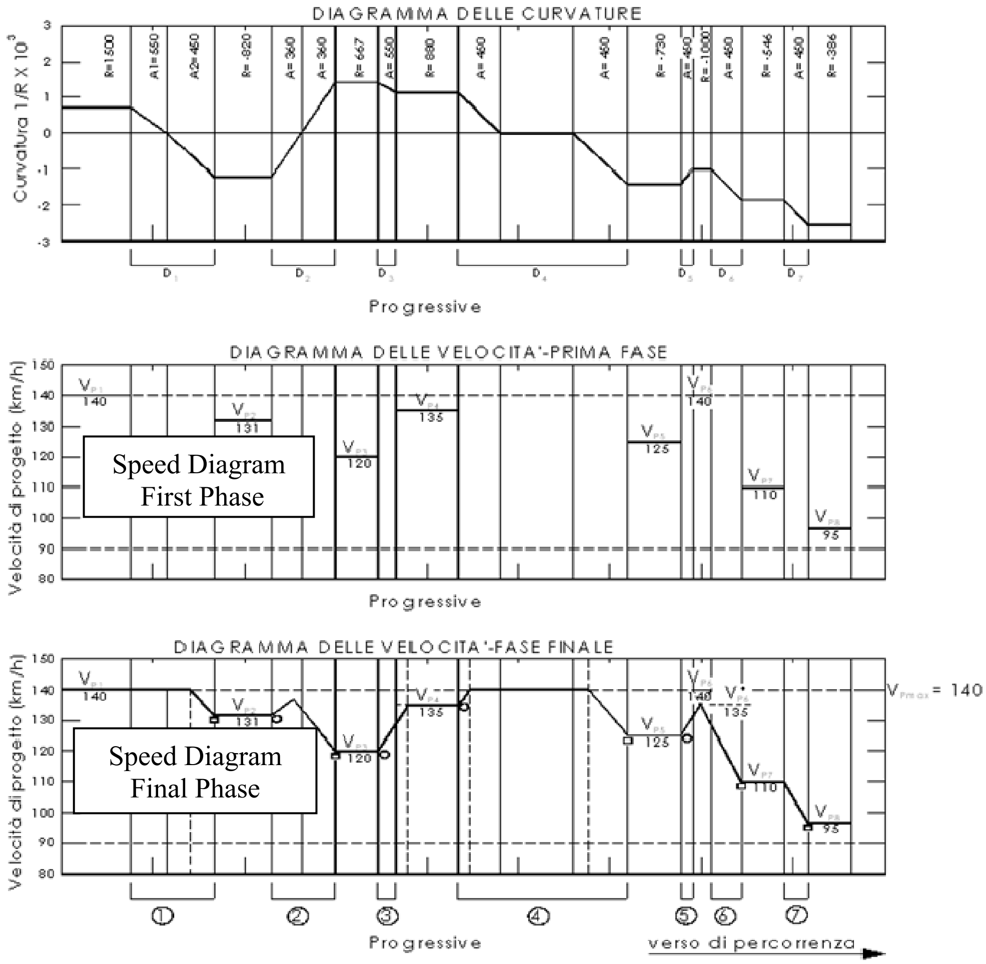
where

ΔV = sequential speed difference ($V_{P1} - V_{P2}$) (km/h);
V_m = mean speed between two elements (km/h); and
a = acceleration or deceleration rate = ±0.8 (m/s^2).

The speed profile is constructed in two phases. In the first one, the speed that corresponds to the safe speed for the specific curve as derived from the vehicle dynamics formula. In the second phase, the transitional lengths for achieving the consecutive speed values based on the 0.8 m/s^2 acceleration–deceleration rates are drawn, paying attention to the maximum attainable speed values and speed differences. The two-phase construction of the speed profile is shown in Figure 12.

After the speed diagram is constructed and the transition distance values checked, the speed consistency has to be verified for both running directions to ensure homogeneous speed gradients. For this purpose the following conditions has to be checked.

- For major roads with design speed $V \geq 100$ km/h (divided arterials and two-lane major and local rural roads), in the transition between elements characterized by $V_{P\max}$ to curves with lower speed, the design speed difference has to be ≤10 km/h. Moreover, between two successive curves the design speed difference has to be never larger than 20 km/h (15 km/h, value suggested).
- For other types of roads with $V_{P\max} \leq 80$ km/h (urban arterials and streets) in the transition between elements characterized by $V_{P\max}$ to curves with lower speed, the design speed difference has to be ≤5 km/h. Moreover, between two successive curves the design speed difference has to be never larger than 20 km/h (10 km/h, value suggested).



①	$V_{Pmax} = V_{P1} > V_{P2}$	$D_1 > D_T$	D_T transition distance to decelerate from V_{P1} to V_{P2}	$V_{P1} - V_{P2} \leq 10$ accett.
②	$V_{Pmax} > V_{P2} > V_{P3}$	$D_2 > D_T$	D_T transition distance to accelerate from V_{P2} to V_{P3}	$V_{P2} - V_{P3} \leq 20$ accett.
③	$V_{P3} < V_{P4} < V_{Pmax}$	$D_3 < D_T$	D_T transition distance to decelerate from V_{P3} to V_{P4}	$V_{P4} - V_{P3} \leq 20$ accett.
④	$V_{Pmax} > V_{P4} > V_{P5}$	$D_4 > (D_{T4} + D_{T5})$	D_T transition distance to accelerate from V_{P4} to V_{Pmax} [illegible]	$V_{P4} - V_{P5} \leq 20$ accett. $V_{Pmax} - V_{P5} > 10$ non acc.
⑤	$V_{P5} < V^{*}_{P6} < V_{Pmax}$	$D_5 < D_T$	D_T transition distance to accelerate from V_{P5} to V_{Pmax}	$V^{*}_{P6} - V_{P5} \leq 20$ accett.
⑥	$V_{Pmax} > V^{*}_{P6} > V_{P7}$	$D_6 < D_T$	D_T transition distance to decelerate from V_{max} to	$V^{*}_{P6} - V_{P7} > 20$ non acc.
⑦	[illegible]	[illegible]	D_T transition distance to decelerate from V_{P7} to V_{P8}	$V_{P7} - V_{P8} \leq 20$ accett.

Accett.= acceptable

FIGURE 12 Example of speed profiling according to the official Italian Ministry for Infrastructure and Transport method.

Due to the global character of this checking, in case the above-mentioned conditions are not verified, it is necessary to redesign the road alignment.

Academic Approach

An operating speed model based on the so-called environmental speed, i.e., a speed that conforms to a specific section of a highway with similar alignment features, was used by Cafiso et al. (2008). The operating speeds (V_{85}) on tangents and curves according to this model can be calculated using a prediction model for two-lane rural roads in Italy developed on a sample of 80 sections of two-lane rural roads in Sicily.

$V_{85i} = 0.987 \times V_{\text{env}} - 0.0418\ \text{CCR}_{si} \times V_{\text{env}}/100$ [km/h]

The environmental speed of road section is modeled as

$V_{\text{env}} = 100.05 - 0.197\ \text{CCR}_{\text{sect}} + 2.147\ W$

Curvature change rate of single element i $\text{CCR}_{si} = \gamma_i/L_{\text{i}}$ [gon/km]

where

γ_i = deflection angle of the geometric element (gon),
W = width of the paved cross section (lanes and shoulders) (m), and
CCR_{sect} = section curvature change rate.

In order to single out the road sections with homogenous horizontal alignment characteristics, reference was made to the indications to be found in the German Guide for the design of roadways. This Guide provides a method for singling out homogenous road sections by evaluating the cumulative section curvature change rate (CCR_{sect}).

$\text{CCR}_{\text{sect}} = \Sigma_i\ |\gamma_i| \ /\ L$ (gon/km)

The sum of the deflection angles [γ_{i} (gon: centesimal degree)] of contiguous elements i of the horizontal alignment is represented in a diagram as a function of distance [L (km)] (Figure 13). It is quite easy to identify the sections in which to subdivide the road. They are represented in Figure 13 by various constant slope lines interpolating the cumulative angle deviation curve. Based on the German procedure, to identify homogenous sections according to CCR values, the minimum length of the sections has to be higher than 2 km. The CCR value (CCR_{sect}), for an identified section, is equal to the slope value of the line representing the section.

Marchionna and Perco (2008) proposed an approach based on the desired speed of a homogeneous road section determined in the same way as illustrated in Figure 13. Speed data were collected on 100 tangent sites of 27 two-lane rural roads and 131 curve sites of 29 two-lane rural roads. At least 100 valid passenger car speeds for each site were collected. The operating speed prediction model development is explained in Perco (2008). The desired speed for a homogeneous section is determined according to the following formula.

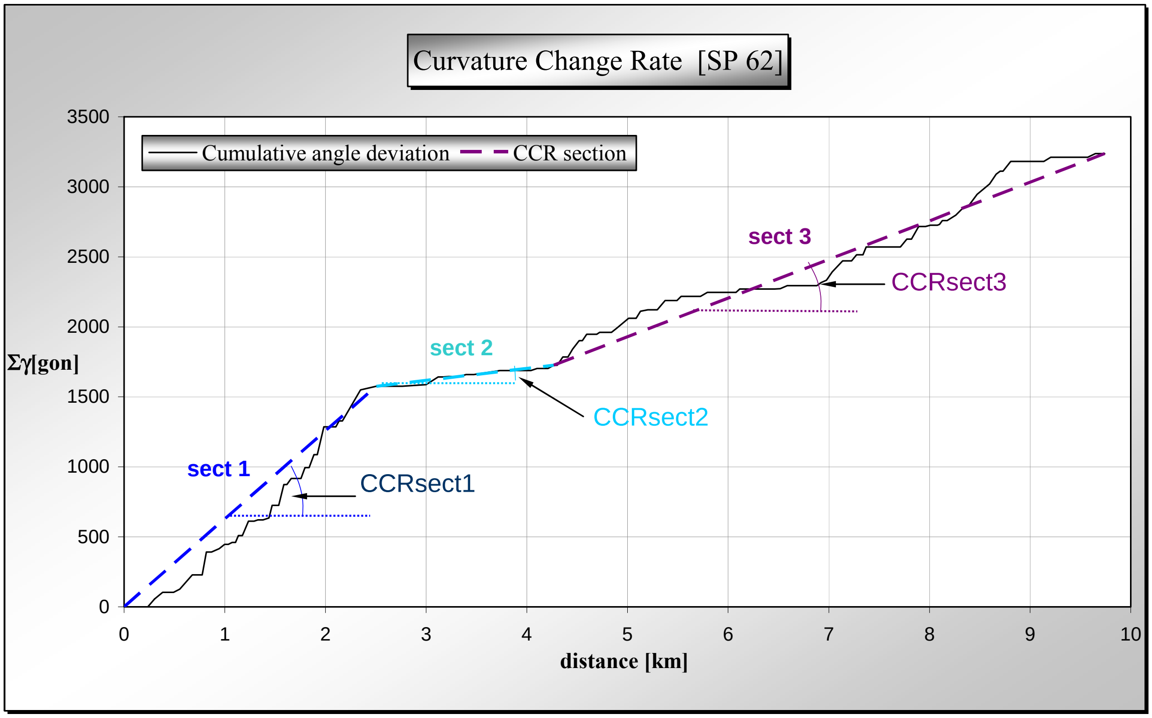


FIGURE 13 Evaluation of homogeneous sections based on the cumulative curvature change rate of the section $\mathbf{CCR_{sect}}$.

$V_{des} = 123.54 - 2.79 \cdot \mathrm{CCR}^{0.47}$ $\qquad R^2 = 0.77$

where

V_{des} = desired speed (km/h);
CCR = curvature change rate (gon/km); and
R^2 = coefficient of determination.

The desired speed according to this model represents the maximum speed attained by drivers on long (independent) tangents of the homogeneous road section. For the individual curves the operating speed is calculated as a function of radius based on a set of equations that correspond to different ranges of CCR values of the homogeneous road section to which the curves belong. These equations are given in Table 23. The diagrammatic illustration of equations in Table 23 is further shown in Figure 14.

TABLE 23 Operating Speed Models for Individual Curves According to Marchionna and Perco (2008)

Range of CCR	Range of Desired Speed Rounded from Eq. 2	Range of Sample Radii	Prediction Equations	R^2
1	2	3	5	6
gon/km	km/h	m	km/h	—
<30	>110	200–2,500	$V_{85} = 124.08 - \frac{563.78}{\sqrt{R}}$	0.40
30–80	110–100	100–635	$V_{85} = 118.11 - \frac{510.56}{\sqrt{R}}$	0.58
80–160	100–90	77–480	$V_{85} = 111.65 - \frac{437.44}{\sqrt{R}}$	0.80
>160	<90	36–300	$V_{85} = 100.85 - \frac{346.62}{\sqrt{R}}$	0.89

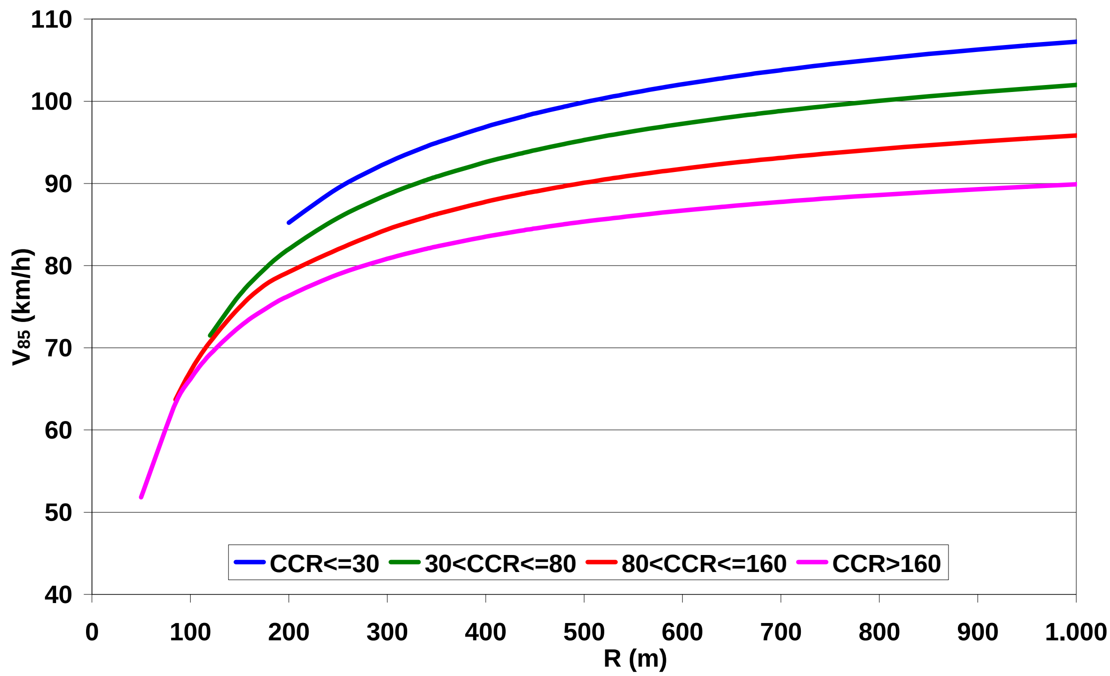


FIGURE 14 Operating speed modeling according to Marchionna and Perco (2008).

To complete the procedure of the speed profiling method the following acceleration and deceleration rates between successive geometric elements were also proposed based on the speed-profiles collected along tangents before 18 curves (deceleration) and after 20 curves (acceleration) (at least 100 vehicles for each site):

$d = 1.757 - 0.222 \cdot \ln(R) \qquad R^2 = 0.74$

$a = 1.328 - 0.159 \cdot \ln(R) \qquad R^2 = 0.45$

where

d = deceleration rate (m/s^2);
a = acceleration rate (m/s^2);
R = curve radius (m); and
R^2 = coefficient of determination.

For the construction of the speed profile authors of IT-5 assume that

- The operating speed is constant through the horizontal curve;
- The acceleration and deceleration occur on tangents and spirals approaching and departing the curve;
- The operating speed on tangents, spirals, and large radius curves tends to the desired speed;
- The operating speed on curve cannot be higher than the desired speed of the road section to which the curve belongs; and
- The operating speed profile must be constructed for both directions because the acceleration rate is not equal to the deceleration rate.

To construct the operating speed profile, the following steps have to be performed.

- The horizontal alignment of the road is divided into sections with a relatively uniform general character.
- The representative for the section CCR is calculated for each road section.
- The desired speed is estimated using the corresponding formula for each road section in function of the CCR of the road section.
- The operating speed on a curve is estimated using equations of Table 23 or Figure 14 with reference to the CCR of the road section to which the curve belongs.
- The operating speed achieved on a tangent (or reverse spiral) is estimated using the acceleration and deceleration rates and as a function of the curve radii at the boundaries of the tangent.

In particular, the tangent can be classified as independent, if the desired speed is achieved, or nonindependent if, on the contrary, the desired speed is not achieved because the tangent is not long enough. Clearly, the critical length L_{LIM}, which represents the minimum tangent length, required to achieve the desired speed depends on the CCR of the road section that affects the desired speed and the operating speeds on curves, and on the radii of the curves at the boundaries that affect the operating speeds and the acceleration and deceleration rates.

$$L_{T1} = \frac{V_{\text{des}}^{\;2} - V_{85-n}^{\;\;2}}{2 \cdot a}$$

$$L_{T2} = \frac{V_{\text{des}}^{\;2} - V_{85-n+1}^{\;\;2}}{2 \cdot d}$$

$$L_{\text{LIM}} = L_{T1} + L_{T2}$$

where

V_{des}= desired speed (m/s);
$V_{85\text{-}n}$= operating speed on curve n (m/s);
$V_{85\text{-}n+1}$= operating speed on curve n+1 (m/s);
d = deceleration rate (m/s^2);
a = acceleration rate (m/s^2);
L_{T1} = transition length between $V_{85\text{-}n}$ and V_{des} (m);
L_{T2} = transition length between $V_{85\text{-}n+1}$ and V_{des} (m); and
L_{LIM}= critical tangent length (m).

For tangent lengths L longer than the critical length (Case 1: $L \geq L_{LIM}$) the desired speed is achieved on the tangent whereas for tangent lengths shorter than the critical length (Case 2: $L < L_{LIM}$) the desired speed is not achieved and the maximum operating speed v^* achieved on the tangent is calculated using equation

$$V^* = \sqrt{\frac{2 \cdot a \cdot d \cdot L + d \cdot V_{85-n}^{\;\;2} + a \cdot V_{85-n+1}^{\;\;2}}{(a + d)}}$$

These different cases are illustrated in Figure 15.

If the tangent (or reverse spiral) length L between two curves is shorter than the transition length needed to vary the speed from the speed of the first curve to the speed of the second curve, then two different cases are possible, depending if the speed of the first curve is lower (Case 3) or higher (Case 4) than the speed of the second curve. In these cases the construction of the speed profile, shown in Figure 15, prevents an operating speed on curve higher than that estimated using equations of Table 23.

Case 3: $V_{85\text{-}n} < V_{85\text{-}n+1}$

$$L < L_{Ta}$$

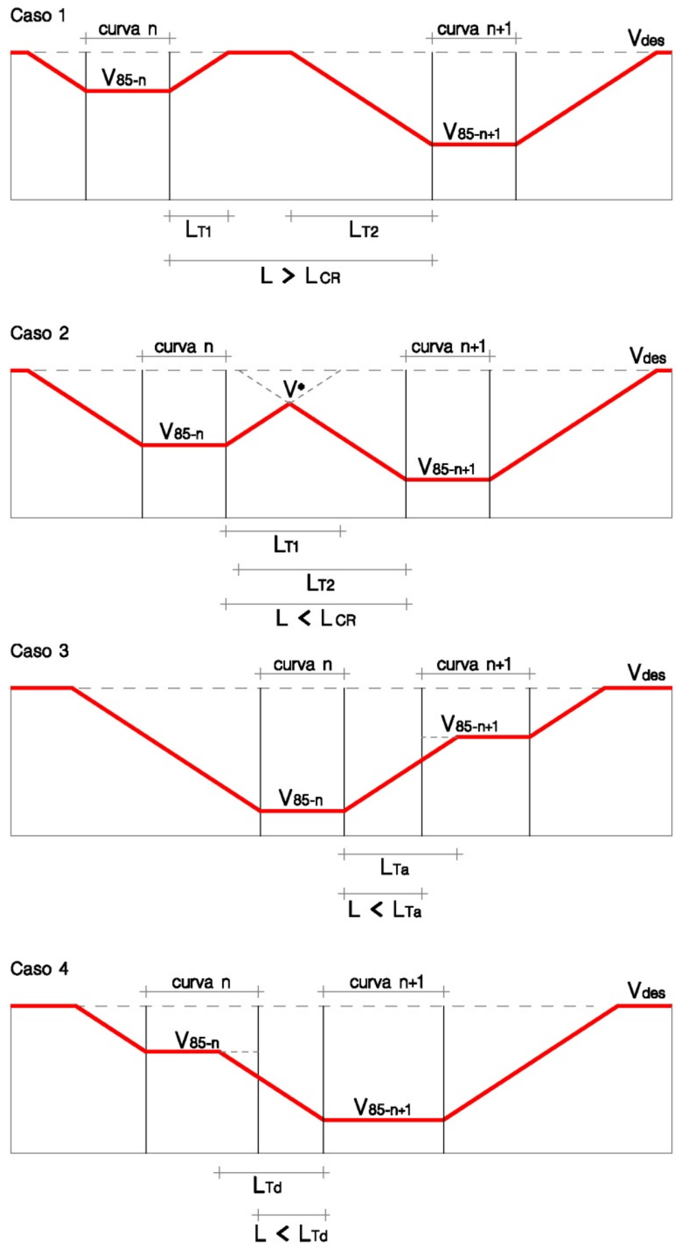


FIGURE 15 Operating speed profiling according to Marchionna and Perco (2008).

$$L_{Ta} = \frac{V_{85-n+1}{}^{2} - V_{85-n}{}^{2}}{2 \cdot a}$$

Case 4: $V_{85\text{-}n} > V_{85\text{-}n+1}$

$$L < L_{Td}$$

$$L_{Td} = \frac{V_{85-n}{}^{2} - V_{85-n+1}{}^{2}}{2 \cdot d}$$

where

V_{85-n} = operating speed on curve n (m/s);
V_{85-n+1} = operating speed on curve n + 1 (m/s);
d = deceleration rate (m/s^2);
a = acceleration rate (m/s^2);
L_{Ta} = transition length between V_{85-n} and V_{85-n+1} for $V_{85-n} < V_{85-n+1}$ (m); and
L_{Td} = transition length between V_{85-n} and V_{85-n+1} for $V_{85-n} > V_{85-n+1}$ (m).

Obviously, case 4 requires particular attention because the driver has to begin the speed reduction inside the first curve. Therefore an adequate sight distance to collect the necessary visual information must be guaranteed. A specific description of all the construction rules of this operating speed-profile model and their comparison with the construction rules of IHSDM was recently presented in Marchionna et al. (2010).

UNITED KINGDOM

Official Approach

In the United Kingdom, the official document for the geometric design is the Design Manual for Roads and Bridges, Volume 6 Road Geometry, Section 1 Links, TD 9/93, Edition 2002. In this manual and generally in the design process the decisive parameter for speed on rural two-lane highways is the design speed. The operating speed issue is not explicitly discussed in the manual although the 85th-percentile speed is associated directly to the fundamental diagram for the determination of the design speed (Figure 16).

In this figure, the parameter alignment constraint A_c for two-lane rural highways is defined as

$$A_c = 12 - \mathrm{VISI}/60 + 2B/45$$

$$\mathrm{VISI} = \frac{n}{\frac{1}{V_1} + \frac{1}{V_2} + \frac{1}{V_3} + \cdots + \frac{1}{V_n}}$$

where

B = highway bendiness (degrees/km);
VISI = harmonic mean visibility (m);
V_i = sight distance at point i = 1, …, n; and
n = number of observations on a minimum 2-km long highway according to Figure 17.

The layout constraint, L_c, in km/h is also determined as shown in Table 24.

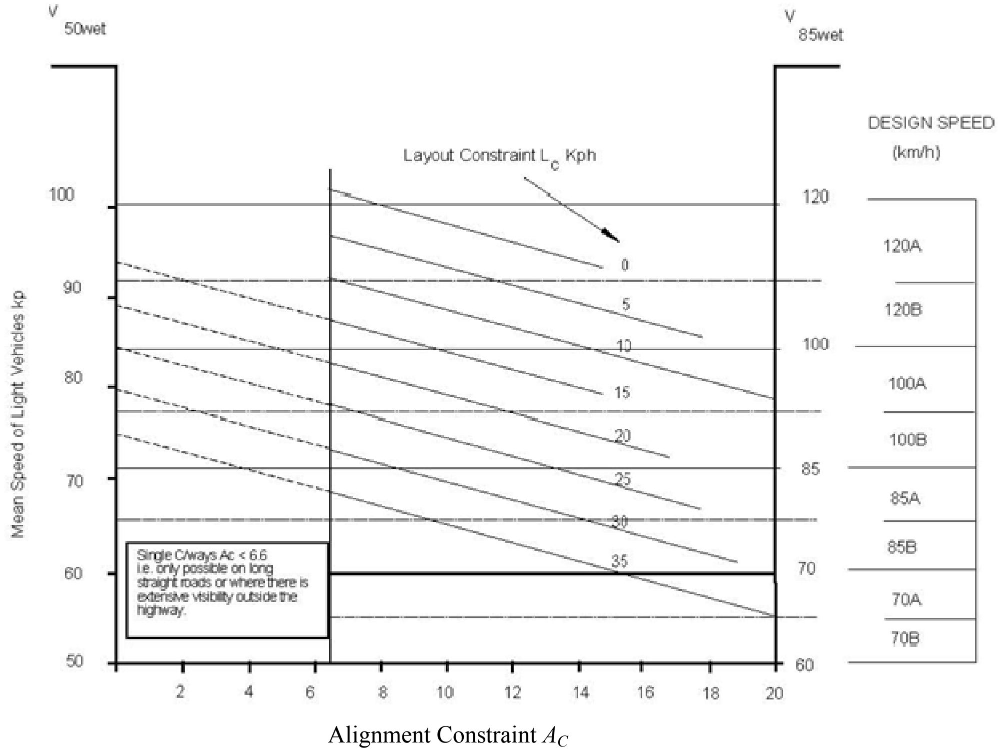


FIGURE 16 Fundamental diagram for determining the design speed of a highway in the United Kingdom.

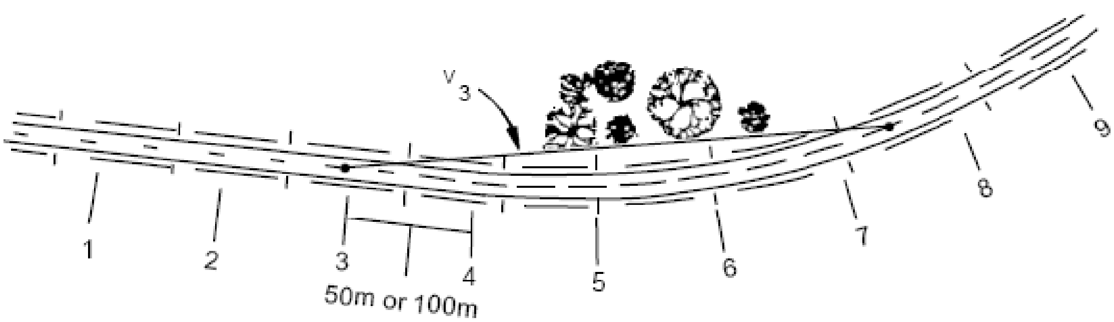


FIGURE 17 Measurement of sight distance to determine the VISI parameter (UK-1).

TABLE 24 Determination of Layout Constraint L_c

Carriageway Width (Ex. Metre Strips)	6m		7.3m	
Degree of Access and Junctions	H	M	M	L
Standard Verge Width	29	26	23	21
1.5m Verge	31	28	25	23
0.5m Verge	33	30		

where

H = number of driveways 9 to 12 per km
M = number of driveways 6 to 8 per km
N = number of driveways 2 to 5 per km

Academic Approach

Despite the fact that the official U.K. road design guide is not using the operating speed concept, academic research has been initiated by Kerman et al. (1982). The following operating speed model was proposed for both divided and undivided highways:

$$V_{85} = V_A \times \left(1 - \frac{V_A^2}{400 \times R}\right)$$

where

V_{85} = operating speed at midpoint of horizontal curve (km/h);
V_A = approach speed or design speed (85th-percentile speed) (km/h); and
R = radius of curve (m).

Due to various shortcomings of this model, it did not find any practical implementation. Another operating speed model for U.K. conditions was also proposed by Bird and Hashim (2005). It is a single-variable version for curves.

$$V_{85} = 104.379 - \frac{4698.216}{R} \qquad (R^2 = 0.79)$$

where

V_{85} = operating speed (km/h) and
R = radius of curve (m).

The best multiple variable model for the 85th operating speed according to Bird and Hashim (2005) for curves is

$$V_{85} = 119.073 - \frac{518.275}{\sqrt{R}} - \frac{125440}{\text{ATL}^2} + \frac{413.181}{\Phi^2} \qquad (R^2 = 0.88)$$

where

V_{85} = operating speed (km/h);
R = radius of curve (m);
ATL = average tangent length (preceding and following the curve) (m); and
Φ = deflection angle (degrees).

For tangents, Bird and Hashim (2005) propose either the single variable model:

$$V_{85} = 82.745 + 0.523 \times \sqrt{L} \qquad (R^2 = 0.45),$$

or the multiple variable model:

$$V_{85} = 95.414 + 0.476 \times \sqrt{L} - 4.824 \times \sqrt{\text{ADC}} \qquad (R^2 = 0.56)$$

where

V_{85} = operating speed (km/h);
L = tangent length (m); and
ADC = average degree of curvature (preceding and following curve), where degree of curvature = 1746.4/(curve radius).

Currently the operating speed concept in the United Kingdom is investigated within a research project of the Highways Agency (2002).

FRANCE

Definition

In France, the operating speed is defined by the conventional 85th-percentile free-flow speed (V_{85}). The French Guide (2002) for the design of highways and freeways provide specific operating speed determination functions for all road functional categories that are used for the design of new roadways. These functions include as independent parameters the horizontal curve radius and the longitudinal slope (gradient) for various numbers and widths of lanes. They were based on measurements carried out before 1986 for undivided roadways and in 1999 for freeways. In specific, the French Guide for undivided and divided non-freeway roadways provides the following functions for the operating speed.

1. For roadways with two-lane 5-m travel area width:

$$V_{85} = {}^{92}\!\big/\!\left(1 + 346/\sqrt{R}\right) \qquad \text{and} \qquad V_{85} = 92 - 0.31G^2;$$

2. For two-lane and three-lane roadways, with travel area widths equal to 6 m and 7m, respectively:

$$V_{85} = {}^{102}\!\big/\!\left(1 + 346/\sqrt{R}\right) \qquad \text{and} \qquad V_{85} = 102 - 0.31G^2; \text{ and}$$

3. For all four-lane roadways (divided non-freeway):

$$V_{85} = {}^{120}\!\big/\!\left(1 + 346/\sqrt{R}\right) \qquad \text{and} \qquad V_{85} = 120 - 0.31G^2$$

where

R = horizontal radius [m]
G = gradient [%]

In the above formula, the influence of gradient applies beyond the first 250 m of the application of the longitudinal slope. The corresponding diagrams for the above functions are given in Figure 18.

Where the alignment is tangent the operating speeds used are as follows.

- Two-lane roadways with a travel area width of 5 m, 92 km/h for a speed limit of 90 km/h;
- Two- and three-lane roadways with a travel area width of 6/7 m, 102 km/h for a speed limit of 90 km/h; and
- Four-lane divided nonfreeways roads 120 km/h for a speed limit of 110 km/h.

The speed limit is always the decisive operating speed value for use in the geometric design of a roadway. The French Guide for the design roadways other than freeways does not provide any further information on determining the speed profile along a specified alignment.

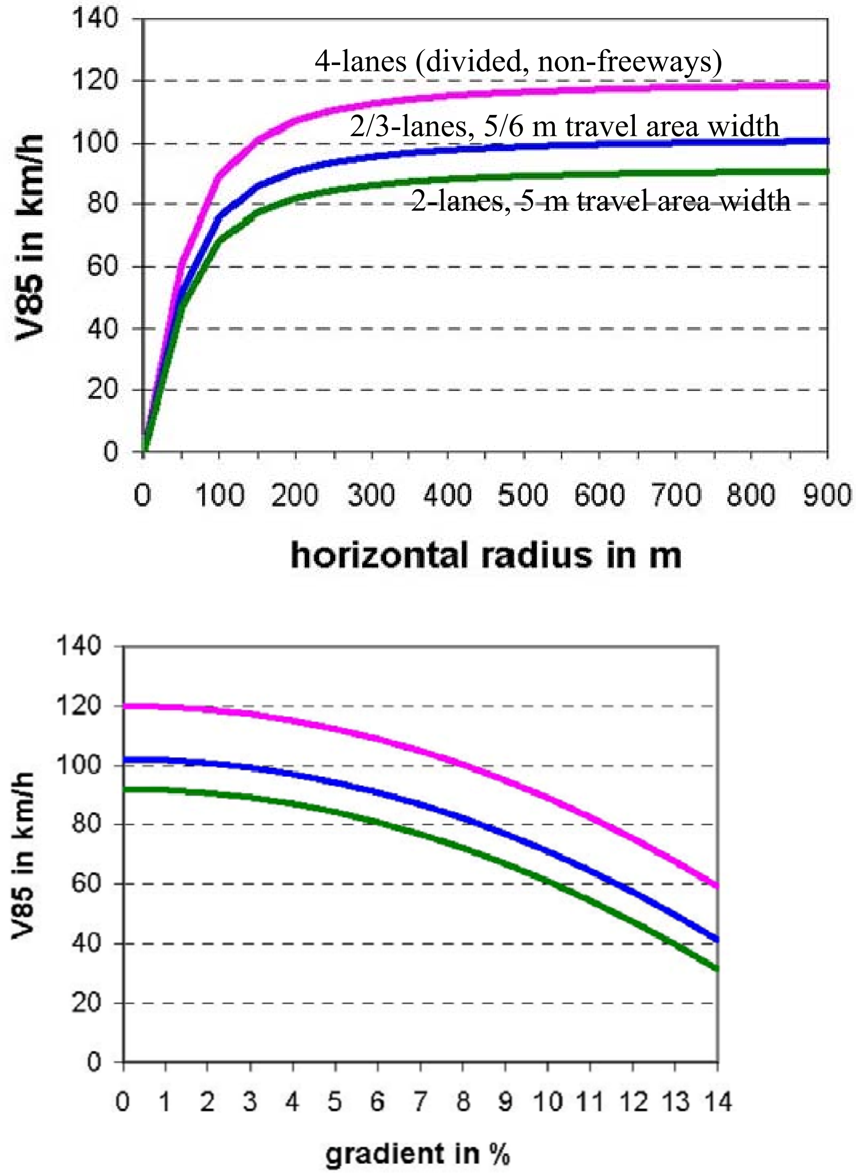


FIGURE 18 Operating speed for undivided and divided non-freeways roads as a function of horizontal radius and gradient according to the French road design guide.

AUSTRIA

Official Approach

In Austria the operating speed corresponds to the 85th-percentile speed of free passenger car flow and is called Project Speed like the case of Switzerland. Its determination and use is described in the *Austrian Guide for the Geometric Design of Highways* RVS 3.23 that goes back to 1997 (*Austrian Guide*, 1997). According to this guide, the operating speed is a function of the horizontal curve radius and the grade of the road and should be determined according to these parameters. Its determination follows from Tables 25 and 26 that are applied simultaneously for every alignment and the smallest values among the two is chosen.

If the smallest value is greater than the legislative maximum speed limit for the corresponding road category according to the Austrian Driving Code, then the speed limit is chosen as the accepted operating speed value. The operating speed profile along a highway alignment that exists in its plan view and profile results by applying the mentioned two tables whenever a horizontal radius or a gradient change results. No further discussion is given as for any smoothing of the resulting in continuities in the speed profile. It is only mentioned that the highway alignment should be selected so that no "abrupt" changes to the profile result but without any quantitative indication of the meaning thereof. Informal information indicates that the future approach of the operating speed issue will be like the one to be followed in Germany, i.e., to determine the highway alignment in a way that makes the operating speed profile obsolete.

GREECE

Official Approach

The Greek Ministry of Infrastructure, Transport, and Networks in the Guidelines for the Geometric Design of Highways (OMOE-X, 2001) distinguishes between roadways with a grade equal to or less 5% and greater than 5%. For the first case the following equation is used for calculating the operating speed (85th-percentile speed) on roadways with a grade ≤5%.

$$V_{85} = \left[10^6 / (10150.10 + 8.529 \times \mathrm{CCR})\right] + [(b - 3.5) \times 20] \qquad (G \leq 5\%)$$

where

CCR = curvature change rate of the single curve (gon/km) and
b = lane width = 3.25/3.50/3.75 m.

TABLE 25 Determination of the Operating Speed V_{85} as a Function of Horizontal Radius in Austria

Radius (m)	30	50	80	130	200	300	400	500	600	800
V_{85} (km/h)		50	60	70	80	90	100	110	120	130

TABLE 26 Determination of the Operating Speed V_{85} as a Function of the Grade

Grade (%)	12	11	10	9	8	7	6	5	4.5	4
V_{85} (km/h)	40	50	60	70	80	90	100	110	120	130

For roadways with a grade greater than 5% and less or equal to 7% the following equation is used:

$V_{85} = 73.260 - 0.015 \times \mathrm{CCR}$ $(5\% < G \leq 7\%)$

While for grades greater than 7% and less than 10% the following equation applies:

$V_{85} = 79.456 - 0.014 \times \mathrm{CCR}$ $(7\% < G < 10\%)$.

In both of the above two equations, it is assumed that they are applied at a distance beyond 250 m from the starting point of the specific grade.

The operating speed model is used to warrant a less or equal to 10km/h or 20 km/h variation in the operating speed between successive design elements for new or existing two-lane rural roads respectively. These models were developed in early 1990s and are still in use.

Academic Research

Based on data from 58 curved sites, Kanellaidis et al. (2000) investigated the relationship between operating speed on curves and various geometric design parameters. The proposed *V*85 operating speed model is

$V_{85} = 129.88 - (623.1/\sqrt{R})$

where *R* is the radius of curve (m).

In 2008 new measurements of the 85th operating speed on two-lane rural highways with a grade less or equal to 5% resulted in the following equations (Xenakis, 2008).

$$V_{85} = {}^{128400.977}\!/_{(\mathrm{CCR} + 1284.010)}$$

for lane widths 3.25 and 3.50 m (insignificant speed change) and

$$V_{85} = {}^{111222.738}\!/_{(\mathrm{CCR} + 994.957)}$$

for lane width equal to 3.75 m.

By comparing operating speeds of the year 2008 and the years around 1991–1992 an increase of about 9 km/h (about 0.5 km/h/year increase) on the tangent (CCR = 0 gon/km) was observed, while this difference diminishes gradually up to the value of CCR = 500 gon/km (about 127 m), beyond of which operating speeds of both time periods coincide.

CHAPTER 4

Speed Models in Other Regions and Road Types

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AUSTRALIA

McLean (1981) studied the effects of horizontal alignment on speeds in Australia. He collected speed data at 120 horizontal curves on two-lane rural highways and on the approach tangents to those curve sites. Moreover, speed data were collected also at 20 sites on level tangent sections with length greater than 1.2 km in the vicinity of curve sites. A minimum of 100 speed measurements were taken at each site. Separate car and truck mean and 85th percentile speeds were determined for each site. Table 27 lists the ranges of traffic and road geometry conditions of the curve sites.

McLean (1981) found that for curves with a design speed lower than 90 km/h, the 85th-percentile speed tends to be higher than the design speed whereas the 85th percentile speeds are generally lower than the design speed when design speeds are higher than 100 km/h. McLean (1981) suggested that the speed at which a driver wishes to travel a road section could have an influence on the speed at which he chooses to negotiate curves along that section. He named this speed as the desired speed of the road section and defined it as the speed at which drivers choose to travel under free-flow conditions when they are not constrained by alignment features. Consequently, a subjective assessment was made of each road on which curves were studied to divide it into sections of relatively uniform character. This assessment considered factors as overall alignment standard, topography, cross section, traffic volumes, adjacent land use, and proximity to major urban development. The lengths of these sections ranged from 3 to 30 km. The higher-value speed distributions measured on each section were regarded as the desired speed pertaining to the section. The data analysis indicated that the desired speed was influenced by road function, typical trip purpose and length for traffic on the road, proximity to urban

TABLE 27 Summary of Traffic and Road Characteristics (McLean, 1981)

Item	Units	Min. value	Max. value	Average value
Car mean speed	km/h	45.3	100.7	72.4
Car 85th percentile speed	km/h	51.0	117.0	83.1
Truck mean speed	km/h	35.6	84.7	62.8
Directional flow	vph	28	704	164
Opposing flow	vph	25	533	146
Radius	m	45	875	146
Superelevation	m/m	0.025	0.140	0.070
Sight distance	m	44	675	190
Seal width	m	5.8	10.3	7.2
Shoulder width	m	0.0	3.7	2.0
Grade magnitude	%	0.0	11.0	3.5

centers and, most importantly for design purposes, by the overall standard of alignment. McLean (1981) proposed Table 28, which gives the typical 85th-percentile desired speeds for the most common road conditions encountered during the research.

The regression analysis of the curve speed data revealed that the observed 85th-percentile passenger car speeds were dominantly influenced by the curve radius and, effectively, by the desired speed pertaining to the road section to which the curve belongs. The other variables listed back in Table 27, with the exception of the sight distance that represented less than 1% of the variability in observed 85th percentile speeds, failed to show a statistically significant effect on curve speeds (p-value 0.05). The regression equation developed was

$$V_{85} = 53.8 + 0.464 \cdot Vd - \frac{3.26 \times 10^{3}}{R} + \frac{8.50 \times 10^{4}}{R^{2}} \qquad R^2 = 0.92,\ p\text{-value} = 0.01$$

However, while this equation was appealing for its simplicity and was very successful in terms of explaining the variability in observed curve speeds, McLean (1981) noted that it tends to produce anomalous results for the extremes of the data range. McLean (1981) stated that this appeared to be due to a nonlinear correlation between the observed values of desired speed and the curvature ($1/R$) for each study site. Therefore, McLean (1981) partitioned the data into four groups according to the desired speed value and performed separate regression analysis for each group that resulted in four linear speed curvature equations. Therefore, the regression coefficients were iterated or extrapolated against the desired speed value to produce the family of curve speed prediction equations given in Table 29 and shown in Figure 19. This family of equations explained a greater proportion of curve speed variability than did Equation 1.

TABLE 28 Typical 85th-Percentile Desired Speeds for Different Road Conditions

Overall Design Speed (km/h)	**Desired Speed km/h**		
	Flat	**Undulating**	**Mountainous**
40–50			70*
50–70		90	
70–90		100	
90–120	115	110	
>120	120		

* Under these conditions, tangent lengths are too short for meaningful measure of "desired speed"; the value given represents the typical maximum 85th percentile speeds measured on available approach tangents

TABLE 29 Speed Models by McLean (1981)

Desired Speed (km/h)	**Speed Models**
60	60–380/R
70	69–715/R
80	77–1050/R
90	85–1410/R
100	95–1960/R
110	105–2920/R
120	115–3940/R

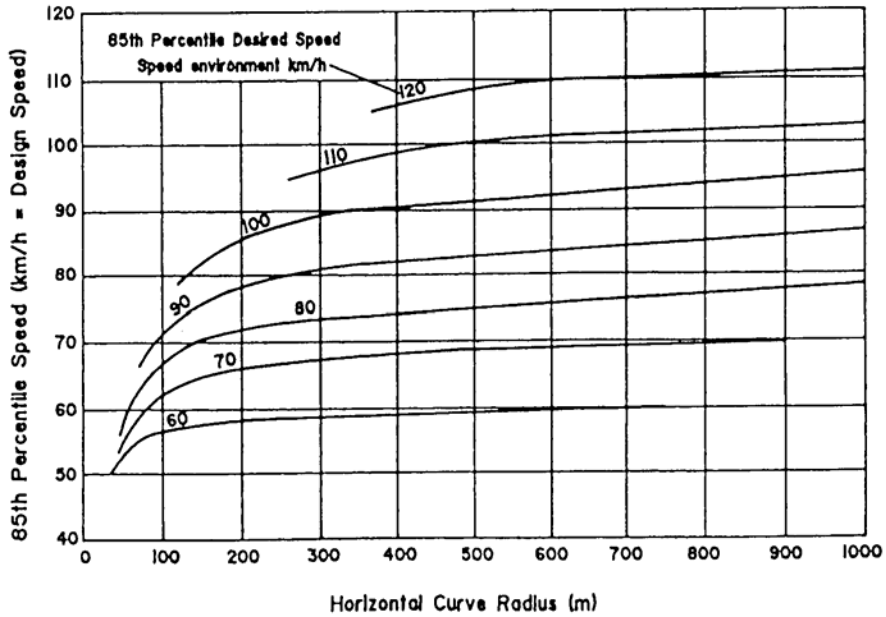

FIGURE 19 Speed prediction on horizontal curves (McLean, 1981).

In response to McLean's (1981) findings, the Australian guideline changed its design procedures for horizontal alignment on lower design speed highways (i.e., a design speed ≤ 100 km/h) to emphasize the importance of the 85th-percentile speeds along an alignment. The Australian design guideline adopted the concept of the "desired speed." In particular, it introduced the term "speed environment" to describe the general character of a road section. The speed environment is numerically equal to the 85th-percentile desired speeds. The guideline provides Table 30 that gives standard values for the speed environment of a roadway. The Australian guideline adopted the same family of prediction equations developed by McLean (1981) to estimate the 85th-percentile speed on curve using the curve radius and the speed environment of the road section to which the curve belongs as independent variables.

TABLE 30 Standard Values for the Speed Environment (McLean, 1981)

Approximate Range of Horizontal Curve Radii (m)	Speed Environment (km/h)			
	Terrain Type			
	Flat	Undulating	Hilly	Mountains
Less than 75			75	70
75–300		90	85	
150–500		100	95	
Over 300–500	115	110		
Over 600–700	120			

JORDAN

Al-Masaied et al. (1995) studied the effects of horizontal alignments on speeds both along curves and tangents in Jordan. Speed data were collected on 57 simple horizontal curve sections and 36 continuous horizontal curves of four primary two-lane rural roads. The simple horizontal curve was defined as a circular curve proceeded by a straight tangent section with a length of at least 800 m. The curve may or may not be accompanied by a transition section. A continuous curve consists of two successive horizontal curves separated by a short tangent with a maximum length of 300 m. The geometric elements included in the analysis were

- Degree of curve DC,
- Deflection angle DF,
- Length of horizontal curve,
- Length of vertical curve within the horizontal curve V_c,
- Gradient G,
- Superelevation,
- Length of spiral,
- Lane width,
- Shoulder width,
- Length of common tangent LT (for continuous curves),
- Prevailing terrain (mountainous, rolling, level),
- Pavement conditions, PC [expressed in terms of present serviceability rating (PSR)],

and

- Posted speed limit.

Free-flow speeds were determined by measuring the time required to traverse a 40-m trap length. The measurements were taken for vehicles with a minimum gap of 6 s. For simple horizontal curves, the measurements were taken along the central part of the curve and on the preceding tangent about 250 m from the beginning of the curve. For continuous curves, measurements were taken at three locations: two measurements were taken along the central part of each curve and the third on the common tangent. The authors adopted the 85th-percentile speed reduction between tangent and curve or successive curves as consistency indicator in this study. The speed data were used to develop statistical models that express the speed reduction as a function of geometric, pavement condition, prevailing terrain, and posted speed variables. The analysis was performed for all vehicle types and separately for passenger cars and trucks. For simple horizontal curves the correlation analysis showed that speed reduction is highly correlated with the degree of horizontal curve, length of vertical curve within horizontal curve, gradient, and pavement conditions. The analysis revealed that lane and shoulder width, superelevation, prevailing terrain, and posted speed had no effect on speed reduction. The degree of the horizontal curve was the most important variable for predicting the speed reduction. The regression equations were listed as follows.

$$\Delta V_P = 3.64 + 1.78 \times \mathrm{DC} \qquad R^2 = 0.51$$

$$\Delta V_L = 2.00 \times \mathrm{DC} \qquad R^2 = 0.69$$

$$\Delta V_T = 4.32 + 1.44 \times \mathrm{DC} \qquad R^2 = 0.42$$

$\Delta V_A = 3.30 + 1.58 \times \text{DC}$ $R^2 = 0.62$

where

ΔV_P = 85th-speed reduction between tangent and curve for passenger cars;
ΔV_L = 85th-speed reduction between tangent and curve for light trucks;
ΔV_T = 85th-speed reduction between tangent and curve for trucks; and
ΔV_A = 85th-speed reduction between tangent and curve for all vehicles.

The authors found that the estimated speed reduction for each type of vehicle is not significantly different from the speed reduction of all vehicles. The authors improved the prediction capability of these equations considering other significant variables. In particular, the variables included were length of vertical curve within the horizontal curve, gradient, and pavement conditions. Since the length of vertical curve within the horizontal curve was found to be highly correlated with gradient, separate models were developed for these two variables. The following regression equations refer to all vehicles.

$\Delta V_A = 1.84 + 1.39 \times \text{DC} + 4.09 \times \text{PC} + 0.07 \times G^2$ $R^2 = 0.77$

$\Delta V_A = 1.45 + 1.55 \times \text{DC} + 4.00 \times \text{PC} + 0.00004 \times V_c^{\,2}$ $R^2 = 0.76$

For continuous curves, the reduction in the 85th-percentile speeds between the first and second curve was modeled as a function of curve geometric variables. The analysis revealed that the radii of the curves had the most significant effect on speed reduction whereas the direction of the second curve with respect to the first one, introduced as dummy variable, which had no effect on speed reduction. The following speed reduction prediction equations were developed for different vehicle types.

$\Delta V_P = \frac{5{,}708}{R_2} - \frac{5{,}689}{R_1}$ $R^2 = 0.72$

$\Delta V_L = \frac{4{,}957}{R_2} - \frac{4{,}888}{R_1}$ $R^2 = 0.77$

$\Delta V_T = \frac{5{,}463}{R_2} - \frac{5{,}463}{R_1}$ $R^2 = 0.66$

$\Delta V_A = \frac{5{,}081}{R_2} - \frac{5{,}081}{R_1}$ $R^2 = 0.81$

The estimated speed reduction might be negative. This occurs if the radius of the first curve is smaller than the radius of the second curve. The authors found also in this case that the estimated speed reduction for each type of vehicle is not significantly different from the speed reduction of all vehicles. The relationship between speed reduction and radii of continuous curve for all vehicles is shown in Figure 20.

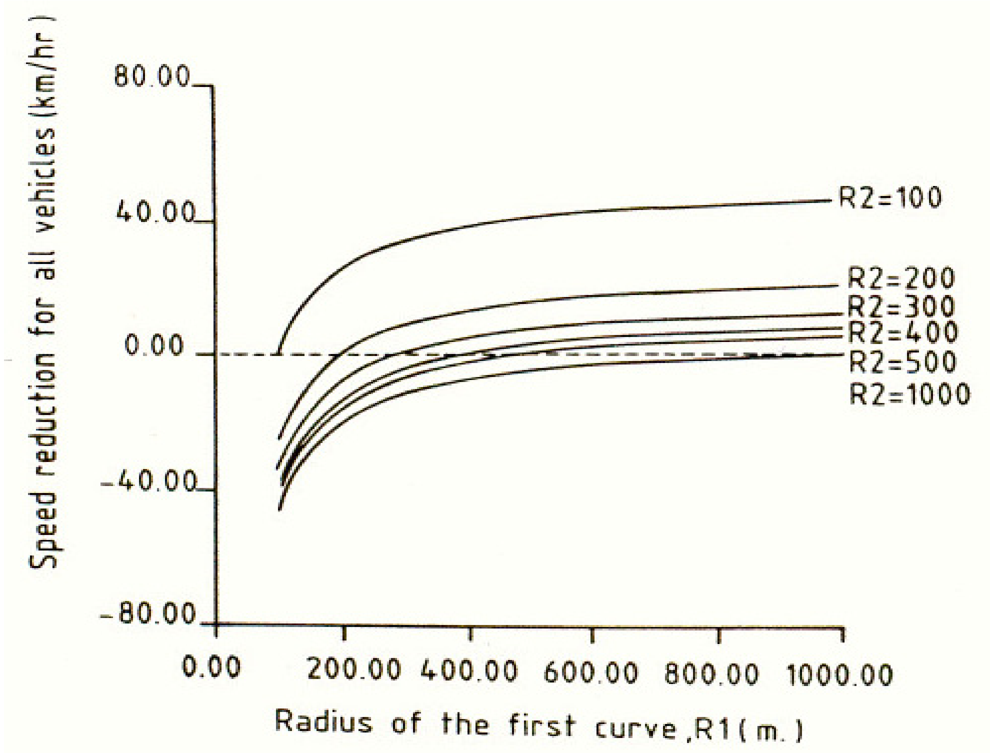

FIGURE 20 Radius versus speed (Al-Masaied et al., 1995).

The analysis revealed that the speed on the common tangent between the two curves was strongly correlated with the length of the tangent, degree of successive curves, and deflection angles. However, the degrees of curves and the corresponding deflection angles were correlated. Therefore, the following equations were developed based on the tangent length and the deflection angles.

$$V_P = 115.00 - \frac{3{,}722}{\mathrm{LT}} - 0.70 \times \left(\frac{\mathrm{DF}_1 \times \mathrm{DF}_2}{\mathrm{DF}_1 + \mathrm{DF}_2} \right) \qquad R^2 = 0.68$$

$$V_L = 106.00 - \frac{3{,}391}{\mathrm{LT}} - 0.73 \times \left(\frac{\mathrm{DF}_1 \times \mathrm{DF}_2}{\mathrm{DF}_1 + \mathrm{DF}_2} \right) \qquad R^2 = 0.71$$

$$V_T = 99.30 - \frac{3{,}099}{\mathrm{LT}} - 0.75 \times \left(\frac{\mathrm{DF}_1 \times \mathrm{DF}_2}{\mathrm{DF}_1 + \mathrm{DF}_2} \right) \qquad R^2 = 0.72$$

$$V_A = 108.30 - \frac{3{,}498}{\mathrm{LT}} - 0.71 \times \left(\frac{\mathrm{DF}_1 \times \mathrm{DF}_2}{\mathrm{DF}_1 + \mathrm{DF}_2} \right) \qquad R^2 = 0.72$$

Although each type of vehicle had its own distinct speed, the length of common tangent and deflection angles had approximately the same effects. Figure 21 shows the relationship between the speed achieved on tangent and its length for different deflection angles.

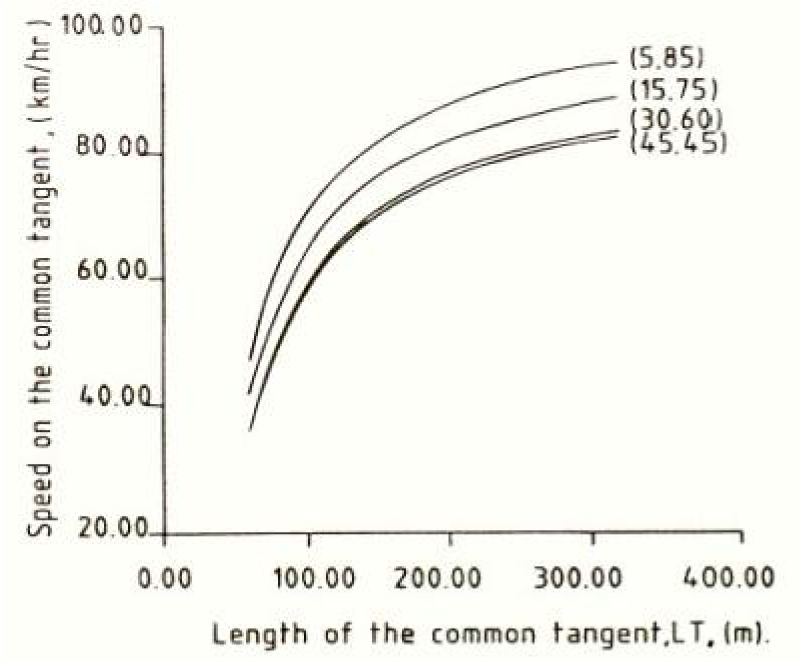

FIGURE 21 Relationship between tangent speed and tangent length for different deflection angles (Al-Masaied et al., 1995).

Similarly, the authors developed the equations based on the tangent length and the degree of successive curves. The equation developed for all vehicles is

$$V_A = 105.47 - \frac{3{,}792}{LT} - 0.27 \times (\mathrm{DC}_1 \times \mathrm{DC}_2) \qquad R^2 = 0.63$$

Figure 22 shows the relationship between the speed achieved on tangent and its length for different degree of successive curves.

The authors proposed a practical application of the equations developed to check the consistency of road alignments. In particular, considering that a good consistent design can be achieved if the speed reduction is less than 10 km/h, the authors used the equations developed for simple horizontal curves to estimate the maximum degree of curve that can be used, also as a function of the pavement conditions and the gradient or the length of the vertical curve within the horizontal curve. For the continuous horizontal curves, the authors proposed Table 31, which shows the limit value of the radius of the second curve in function of the radius of the first curve to ensure a speed difference of ±10 km/h.

VENEZUELA

Andueza (2000) studied the effects of horizontal alignment on speeds, both along curves and tangents, in Venezuela. The speed data were collected with radar on 21 curves and 18 tangents in both directions of the Merida–Botanques section of the Venezuelan Andean highway. The highway is a two-lane rural mountain road with 3.65-m lanes and 1.20-m paved shoulders with a

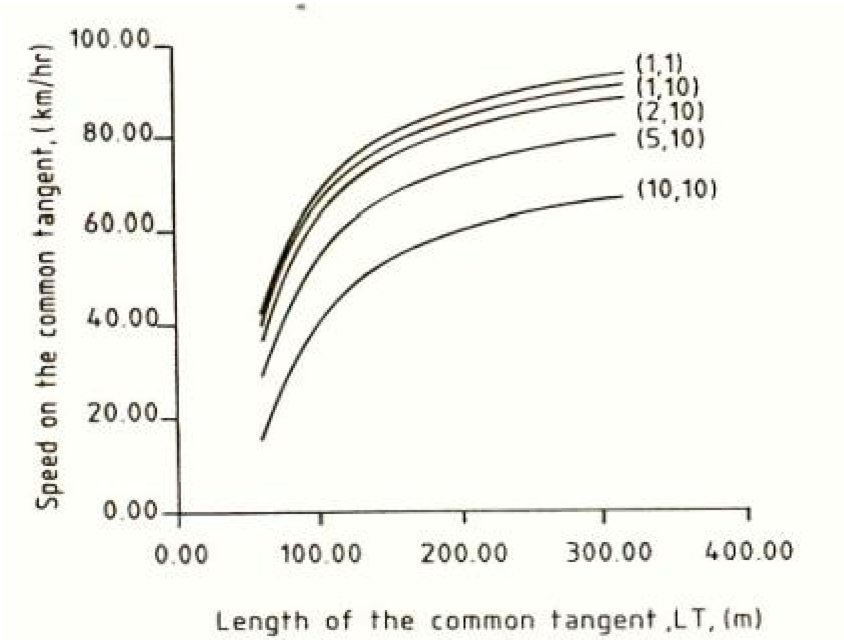

FIGURE 22 Relationship between speed on tangent and tangent length for different degree of successive curves (Al-Masaied et al., 1995).

TABLE 31 Determination of Curve Radius (Al-Masaied et al., 1995)

Radius of the First Curve (m)	Passenger Cars		All Vehicles	
	Radius of the Second Curve (m)		Radius of the Second Curve (m)	
	Maximum	Minimum	Maximum	Minimum
100	122	85	124	84
200	309	148	330	144
300	630	197	732	189
400	1352	236	1880	224
500	4018	267	No limit	252
600	No limit	293	No limit	275
700	No limit	315	No limit	294
800	No limit	334	No limit	311
900	No limit	350	No limit	325
1000	No limit	360	No limit	337

longitudinal grade less than 3%. Only free-flow passenger cars with a time separation from the preceding or following vehicles more than 6 s were selected. Sample size fluctuated between 30 and 64 passenger cars per site. The following independent variables were considered in the analysis.

- Curve variables:
  - Curve radius R_c,
  - Superelevation P,

  - Deflection angle Δ,
  - Minimum sight distance of the curve S,
  - D = S/250 taking $D \leq 1.000$,
  - Previous tangent length L_a,
  - Radius of the previous curve R_a,
  - Length of the tangent before the previous curve L_t, and
  - Radius of the curve preceding the previous curve R_t; and
- Tangent variables:
  - Tangent length L_a,
  - Radius of the following curve R_c,
  - Radius of the previous curve R_a,
  - Length of the tangent before the previous curve L_t, and
  - Radius of the curve preceding the previous curve R_t.

The variable D was adopted as representative for visibility; 250 m is the minimum sight distance required by Venezuelan standard for a speed of 150 km/h. The maximum limit D = 1.000 was adopted because it was estimated that from a certain sight distance the marginal effect on speed would be none. In addition, the curve radius R_c ranged between 130 and 3,970 m whereas the tangent length L_a ranged between 55 m and 555 m. The transitions before the curves can be circular or spiral. Therefore, in order to standardize the measurement, the author included in the tangent length the first part of the spiral length up to the point where deviation with respect to the tangent extension is 1.20 m. The data analysis developed linear regression models to estimate the average and the 85th-percentile speeds on curves and tangents. The most significant variable to estimate curve speed was the radius R_c whereas to estimate the tangent speed was the radius of the previous curve R_a. The models developed are

$$V_{85c} = 98.25 - \frac{2795}{R_c} - \frac{894}{R_a} + 7.486 \times D + 9.308 \times L_a \qquad R^2 = 0.84,\ R_a^2 = 0.82$$

$$V_{mc} = 87.78 - \frac{2251}{R_c} - \frac{739}{R_a} + 5.081 \times D \qquad R^2 = 0.86,\ R_a^2 = 0.85$$

$$V_{85r} = 100.69 - \frac{3032}{R_a} + 27.819 \times L_a \qquad R^2 = 0.79,\ R_a^2 = 0.78$$

$$V_{mr} = 87.65 - \frac{2064}{R_a} + 17.353 \times L_a \qquad R^2 = 0.73,\ R_a^2 = 0.71$$

All the coefficients of the equations are statistically significant at 5% with exception of the tangent length before the curve L_a in the third equation, whose level is 10%. According to field data, the author estimated that the models can be reliable for R_c and R_t values over 100 m; the D variable can be used between 0.30 and 1.00. There are no limitations for L_a in terms of lower value whereas for upper limit, the author stated that values up to about 600 m can be used. The author proposed to use the equations developed to construct 85th-percentile speed-profile to evaluate the alignment and to avoid rough speed changes.

PAKISTAN

Qureshi et al. (2005) collected speed data on three sections of an existing old alignment along the Kotri–Thatta road. The alignment elements of the road were not available. Subsequently, a detailed survey has been conducted. The sites did not present adjacent intersection and physical features that may affect the normal driver behaviour. All sites had the pavement in good conditions. Table 32 summarizes the characteristics of the sections surveyed.

Continuous speed-profile data was collected using a vehicle equipped with GPS capable of taking measurements 20 times in a second. The parameters recorded were distance, lateral acceleration, longitudinal acceleration, profile of test section, plan of test section, etc. Speed data was collected using 25 test drivers that drove each section in both directions. Moreover, speed data was collected following 25 light vehicles along each section in both directions. To ensure that the measured speeds represented the free-flow speeds a time gap between two successive vehicles greater than 5 s was selected. Therefore, 100 speed measurements were taken at each site for both directions. All speed measurements were made during daytime with good weather conditions. Some spot speeds were also measured using a radar gun in order to verify the speed measurements made by the instrumented vehicle. The statistical analysis showed that the normal distribution described satisfactorily the speed distributions at the midpoint of curves. No statistically difference ($p = 0.05$) was observed between speeds of the two directions. The authors performed a linear regression analysis between the 85th-percentile speeds on horizontal curves and the road characteristics for the three road sections separately. For the road sections I and II regression equations containing the radius of curve only as independent variable were developed. For road section III the authors included in the equation also the deflection angle and the length of curve as independent variables. Table 33 shows the linear regression equations developed. In the conclusions the authors underlined the importance to use the operating speed instead of the design speed (in Pakistan AASHTO guidelines are used in practice) in the alignment design process because the design speed calculated for the curves observed was found smaller than the corresponding operating speeds.

TABLE 32 Summary of Study Area (Qureshi et al., 2005)

	Section I	**Section II**	**Section III**
Total length	6 km	5 km	6 km
Number of curves	8	7	16
Radius (m)	150 – 550	170 – 850	110 – 900
Deflection angles (deg)	10.42 – 68.75	5.39 – 47.19	1.35 – 104.2
Length of curve (m)	90 – 180	80 – 280	20 – 200
Gradient (%)	–0.9 to +1.22	–0.99 to +1.22	–7.0 to +3.63
Tangent length (m)	210 – 1210	10 – 1160	30 – 859
Width of carriage (m)	6.5 – 6.0	6.5 – 6.0	6.5 – 6.0

TABLE 33 Summary of Equations Developed by Qureshi et al. (2005)

Road Section	Regression Equation	R^2
I	$V_{85} = 60.0 + 0.0551 * R$	0.98
II	$V_{85} = 68.8 + 0.0405 * R$	0.85
III	$V_{85} = 53.4 + 0.0289 * R - 0.446 * I_c + 0.27 * L_c$	0.81

CHAPTER 5

Deficiencies in Existing Speed Models

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This section describes the main limitations and deficiencies identified in the existing speed profile models.

ISSUES WITH DATA COLLECTION

Study Sites Preclude Some Conditions

In a review of existing speed models, Hassan (2004) noted that the criteria used to select curve sites for most models limited their applicability to horizontal curves with no intersections or change in the number of lanes. Nie and Hassan (2007) similarly stated that the selected study sites of previously developed speed models generally precluded some unfavorable conditions, such as the presence of intersections or change in number of lanes. As a result, the applicability of the developed models is limited due to the simplification process. According to Hassan (2004), more than 50% of the surveyed curves on randomly selected road segments in Eastern Ontario were coupled with intersections or turning lanes. Therefore, most of the developed models may not be applicable to more than half of the aforesaid road segments. In addition, most studies do not address spiral transition curves or compound–reverse curves.

Limited Sample Size and Number of Observations

Hassan (2004) stated that the size of the sample used in model development in terms of number of sites and observations per site is an important issue in examining the validity of an operating speed model. This information is not available for nearly half of the existing models. In addition, although the model developers have generally used large samples in terms of number of sites, these sites cannot possibly represent sites with unique geometric features. Normally, designers use a limited number of radii in a specific highway project or even jurisdiction. Such a lack of unique data points may artificially improve the coefficient of determination (R^2) because of the increase of degrees of freedom. In referring to previously developed models, Misaghi and Hassan (2005) commented that, "…the number of observations for each curve ranged from 30 to 100. The accuracy of such a small sample in representing the 85th percentile speed on the site might be questionable" (Misaghi and Hassan, 2005). While Nie and Hassan (2007) stated that models they developed "can provide a good, quantitative picture on the relationship between different road geometric elements and driver speed choice," they also cautioned that "readers should use these models with some degree of caution because of the relatively small sample size in terms of number of sites and number of observation on each site" (Nie and Hassan, 2007).

Bias–Error from Manual Speed Measurements

In reviewing existing operating speed models, Misaghi and Hassan (2005) noted that, *"…*the data collection device used to record vehicle speed was, in most cases, a manually operated radar gun.*"* They state that manual speed measurements can introduce bias–errors, such as

- Cosine error induced by deviation between the reading beam of the radar/LIDAR gun and the actual driving direction on a curve;
- Human error in measuring speeds; and
- Drivers might change their driving tendency upon perceiving test equipment as speed enforcement.

Nie and Hassan (2007) suggest that some prediction models are questionable due to the bias or human errors induced by manual speed measurements. They note that in a Canadian study, a statistically significant drop of speed averaging 7 km/h was reported when a radar gun was being used (Hassan, 2004). Hassan (2004) also noted that a concern with existing models can be the lack of details necessary to check the validity and applicability of a specific model (such as data collection procedure and sample size). Although researchers exerted every effort to hide the observers, there is a practical limit to how hidden these observers can be. Also, the further away an observer is from the road (necessary to be hidden), the greater the error in speed measurement (such as the cosine error in radar/lidar guns). Such a potentially dominating factor as perception of speed enforcement might affect the speed observations and conceal other factors that would normally influence drivers' speed selection. Factors such as length and urgency of trip or driver's familiarity with the road and level of speed enforcement may be impossible to account for but might dominate the driver's choice of speed in the absence of perceived speed enforcement. The speed database by Misaghi (2003), in which no perceived speed enforcement was present, showed a much weaker relationship between $V85$ and R than the speed observed in most other studies (Hassan, 2004).

Speed Measurement Locations

McFadden and Elefteriadou (2000) commented that the use of two (i.e., midpoint of approach tangent and midpoint of horizontal curve) versus nine data collection locations (curve PC, QP, MP, 3QP, and PT, plus four locations along the approach tangent) for operating may not capture the maximum and minimum speed locations for most sites. The selection of two positions may be valid, but the midpoint of the approach tangent and midpoint of the horizontal curve may not be the appropriate locations for data collection. The data collection location influences the ability to predict speeds on a horizontal curve.

Data Collection Approach

For some studies data were only collected at sites that matched certain criteria. Fitzpatrick and Collins (2000) stated that an advantage to that approach is that greater confidence is possible for knowing which elements affect speed along a roadway. However, a disadvantage occurs when attempting to determine the changing speed along a complex alignment that includes curves that do not fit the criteria developed for the data collection efforts.

UNREALISTIC ASSUMPTIONS OF DRIVER BEHAVIOR

Acceleration and Deceleration Occur Only on Tangents

Many models assume that acceleration and deceleration occur only on tangents (i.e., that speed is constant throughout a curve), which is generally not the case. Nie and Hassan (2007) stated that current speed profiles are constructed based on assumptions that might not be realistic. For example, the speed profiles suggested by Leisch and Leisch (1977) presumed that acceleration and deceleration only occurred on tangents, and that a constant running speed was maintained on the curve. Similar assumptions were made in the work by Ottesen and Krammes (2000) and Fitzpatrick and Collins (2000). However, field data reported by some researchers suggest that drivers likely adjust their speed within the limits of the curved section. For example, according to the models developed by Figueroa and Tarko (2005), only 65.5% of the deceleration transition and 71.6% of the acceleration transition took place on the tangents before and after the curve (Nie and Hassan, 2007). In addition, Misaghi and Hassan (2005) noted that the tendency to adjust speeds on the horizontal curve contradicts the assumption that drivers maintain a constant speed on the curve in most previous research that considered speed reduction in terms of Delta$V$85. Nonetheless, it is consistent with the findings of McFadden and Elefteriadou (2000) and Hassan (2003). Fitzpatrick and Collins (2000) also found that acceleration and deceleration are usually observed within the limits of the curve, whereas the speed-profile model assumes that all acceleration and deceleration takes place before or after the horizontal curve.

Acceleration and Deceleration on Vertical Alignment or Vertical–Horizontal Combination Is Same as on Horizontal Alignment

It was stated by Fitzpatrick and Collins (2000) that "the models developed to predict acceleration and deceleration rates were based on data collected at horizontal curves. Thus, no data were available to predict acceleration and deceleration rates for vertical alignment or for combinations of horizontal and vertical alignment. In these cases, the maximum value selected for horizontal curves on grades is assumed."

Speed Relationships for All Crest and Sag Vertical Curve Types Are Similar

Fitzpatrick and Collins (2000) collected data only for Type I (crest) and III (sag) curves, with an assumption that all crest curves have a speed relationship similar to Type I and all sag curves have a speed relationship similar to Type III. This may or may not be true.

DIFFICULTIES IN ESTIMATING SPEED CHANGES BETWEEN GEOMETRIC ELEMENTS

Calculating Speed Differential from Tangent to Curve (Use of 85th-Percentile Speeds to Estimate Speed Reduction)

Estimating the speed differential from an approach tangent to a horizontal curve is primarily a means to evaluate the design consistency of a highway. Three main methods to estimate this speed differential are documented in the existing models:

- Delta*V*85,
- Delta*V*85, and
- 85MSR.

The Delta *V*85 method used in the majority of existing models, e.g., Lamm et al. (1987), Krammes et al. (1995), Fitzpatrick and Collins (2000), Fitzpatrick et al. (2000a), IHSDM (2003), assumes that speed distribution on successive elements is the same. Hirshe (1987) hypothesized that the use of 85th-percentile speed for evaluating design consistency (i.e., the Delta*V*85 method) tended to underestimate speed reduction experienced by individual drivers. Subsequent research by McFadden and Elefteriadou (2000) validated Hirshe's hypothesis (McFadden and Elefteriadou, 2000). Misaghi and Hassan (2005) further supported such conclusions: "…the current approach to calculate the speed differential is based on calculating the operating speed of the drivers on the curved and the tangent sections and then, subtracting these two values and naming it as the speed differential value. However, this methodology is based on assuming that speed distribution on the successive elements is the same; an assumption that is not necessarily accurate (Hirsh, 1987; McFadden and Elefteriadou, 2000). According to these two studies, speed distributions at the curved and tangent sections are not the same, and thus the simple subtraction of the operating speed values should not be performed. Also, even if the speed distributions are the same, the 85th-percentile driver needs not to be the same in the two locations" (Misaghi and Hassan, 2005).

Nie and Hassan (2007) add that "speed reduction in terms of $\Delta V85$, calculated by simple subtraction of 85th percentile speeds on the tangent and curve, is widely used in design consistency evaluation has been criticized by many researchers." Accordingly, new parameters were introduced based on the speed reductions of individual drivers such as 85MSR (85th percentile of the maximum speed reduction for individual drivers based on data from nine points on the curve and approach tangent) and $\Delta 85V$ (85th-percentile speed reduction for individual drivers based on data from two points on the approach tangent and at the middle of curve). Park and Saccomanno (2006) explained the reason of the underestimation by $\Delta V85$ from a theoretical perspective. However, because of the smaller number of data points used in calculating $\Delta 85V$, it may underestimate the speed reduction relative to 85MSR if the minimum operating speed occurs before or after the midpoint of the curve (Nie and Hassan, 2007). The weakness with the 85MSR models is that they cannot produce a speed profile. The reduction of speed into the curve and increase in speed out of the curve may occur over different lengths (McFadden and Elefteriadou, 2000).

In summary,

- Delta*V*85 can underestimate the speed reduction from tangent to curve;
- The small number of data points used to determine Delta*V*85 in some studies (e.g., just the midpoint of tangent and midpoint of curve) may underestimate speed reduction if the minimum operating speed occurs before or after the midpoint of the curve;
- Even if using only two data points to determine Delta*V*85 is valid, the midpoint of tangent and midpoint of curve might not be the appropriate locations; and
- 85MSR and Delta*V*85 models cannot produce a speed profile.

Quantifying Speed Increases Departing Horizontal Curves

There is no specific measure available to date to quantify the speed increase when departing a horizontal curve. (Nie and Hassan, 2007)

LACK OF UNIFORMITY ACROSS MODELS

Hassan (2004) stated that, "…the reliability of V_{85} or Delta*V*85 prediction is a concern. The fact that there have been such a large number of models that use different predictors to estimate the same parameter appears to indicate a lack of uniformity in these models. Such a lack of uniformity may even be observed within the same country. Misaghi and Hassan (2005) also addressed this issue: "…in the past 50 years, several models have been developed to predict the operating speed at the curved sections. However, the model format, independent variables, and regression coefficients are, in most cases, substantially different from one model to the other. This might have been the result of differences in driver behavior from one locale to the other, and it highlights the fact that no single model is universally accepted."

CONSIDERATION OF ONLY PASSENGER CARS

Most models developed to date focus on passenger car speeds. However, trucks and recreational vehicles may be affected differently than passenger cars by combinations of horizontal and vertical alignment (Fitzpatrick et al., 2000a). Misaghi and Hassan (2005) also recognized that, "…the majority of the models (25 out of 27) were developed concentrating on speed prediction for passenger cars, while models that can predict the speed for light trucks or heavy trucks are lacking." The main obstacle in the development of models for trucks is the insufficient amount of field observed truck speeds (Andolini, Minnicino, and Elefteriadou, 2004). As noted by Andolini, Minnicino, and Elefteriadou (2004), minimum truck speeds may not occur at the same locations as minimum passenger car speeds: "…when evaluating trucks, it is important to consider the performance implications of gradient of vertical alignment. Upgrades on vertical alignment tend to reduce truck speeds more so than passenger cars" (AASHTO, 2001). When identifying alignments that may cause large speed differentials between cars and trucks, it is important that the differential is not underestimated by predicting the truck speed at a point that does not match the location of the true minimum truck speed. McFadden has suggested that speeds be predicted at 13 locations along an alignment to avoid this occurrence (McFadden and Elefteriadou, 2000)."

The speed behavior of motorcycles was studied by Perco (2008). In particular, the study evaluated the possibility to develop a motorcycle operating speed prediction model analyzing the relationships between the motorcycle operating speed and the passenger cars operating speed. Effectively, these two speeds were found to be correlated in urban areas and a prediction model to estimate the motorcycle operating speed starting from passenger car operating speed was developed for tangents of urban roads.

LIMITATIONS OF LINEAR REGRESSION

The majority of existing operating speed models uses conventional linear regression models. Limitations of linear regression models may include the following.

Flawed Assumption of Data Independence

One theoretical deficiency of conventional speed consistency measures, i.e., single-level linear regression models, is underestimation of the speed differential due to the flawed assumption of data independence (i.e., the pseudo-replication fallacy) (Park et al., 2010). Park and Saccomanno (2006) pointed out that although conventional models assume that the speed measures from the upstream and downstream highway sections are independent, this assumption is often violated and causes the speed differential to be underestimated. Park and Saccomanno (2006) argued that an individual vehicle's operating speed on the downstream highway section (e.g., the curved section) could not be independent of the vehicle's operating speed on the upstream highway section (e.g., the approach tangent section). The data collected from the two highway sections are therefore intracorrelated. The assumption that the speed measures obtained from the upstream and downstream highway sections are independent when, in reality, they are intracorrelated violates a key statistical assumption. Making statistical inferences based on a flawed assumption of data independence is an example of the fallacy of pseudo-replication. Pseudo-replication can have a significant impact, leading either to an underestimation or to an overestimation of a study's findings. In speed consistency studies, pseudo-replication has resulted in conventional models underestimating the speed differential from highway section to highway section (Park and Saccomanno, 2006; Park et al., 2010)

Loss of Information Due to Data Aggregation

Another theoretical deficiency of conventional speed consistency measures is the inflation of the adequacy of the model's explanation due to the use of aggregate data, i.e., aggregation bias (Park et al., 2008). When linear regression is used on a descriptive statistic obtained through data aggregation, such as $V85$, it reduces the total and nature of variability associated with the regression function as demonstrated by Tarris et al. (1996), so the influence of geometric elements may be overstated or understated (Hassan, 2004). Conventional studies have relied on information from aggregate data to derive their estimates of the mean speed and 85th-percentile speed of highway sections, and, consequently, their estimates of the speed differential (Delta$V85$) between highway sections (e.g., Lamm et al., 1988; Collins and Krammes, 1996; Fitzpatrick and Collins, 2000; McFadden and Elefteriadou, 2000; Fitzpatrick et al., 2005; Misaghi and Hassan, 2005). Inappropriate model interpretation resulting from loss of information because of data aggregation is an example of ecologic fallacy. Data aggregation may also artificially inflate the adequacy of explanation of the model, i.e., inflate the coefficient of determination (Park and Saccomanno, 2006). Kweon and Kockelman (2005) also noted that, "…spatially and temporally aggregated speeds are subject to an 'ecological fallacy.' Data correlation at aggregate levels can easily differ from that at the individual or disaggregate level. Therefore, researchers should strive to rely on disaggregate data whenever possible."

Using Equations Beyond Their Limits

Questions are raised about the validity of the predicted values when the regression equation is used beyond the range of the collected data (Fitzpatrick and Collins, 2000).

Matching of One Regression Equation to Another

A concern is how well regression equations that are developed with different sets of data interact (Fitzpatrick and Collins, 2000).

LIMITED APPLICABILITY OF MODELS

Most Models Do Not Consider the Combination of Horizontal and Vertical Alignments

Misaghi and Hassan (2005) found that "…the majority of the models deal with the horizontal alignments only, while a recent study by Hassan et al. (2000) showed that considering the effect of two-dimensional (2-D) alignments only, instead of the actual 3-D alignment may dramatically overestimate or underestimate the actual speed values." Andolini et al. (2004) identified a similar limitation in earlier truck speed models: "…in Donnell et al. (2001), regression models to predict 85th percentile truck speeds did not consider the impact of combinations of vertical and horizontal curvature. The research recommended that future research include sag and crest vertical curves (limited sight distance and non-limited sight distance), in combination with a good range of horizontal alignments." To address the limitation, Andolini et al (2004) developed models that do consider such combinations.

Fitzpatrick et al. (2000a) developed models that do consider the combination of horizontal and vertical alignments, but noted as part of their review of existing models that "…current design–speed-based or operating speed-based methods to ensure design consistency are oriented toward horizontal alignment. There is no model to measure design consistency on combined horizontal and vertical alignments. There are also no statistical models to estimate operating speeds on combined alignments. Furthermore, in the United States, the operational effects of combined horizontal and vertical alignment have not been studied." In addition, acceleration–deceleration rates for vertical and vertical–horizontal alignment are lacking and most existing acceleration–deceleration models are based on data collected at horizontal curves only (Fitzpatrick and Collins, 2000).

Relatively Few Tangent Speed Models

Most speed models either did not include tangents or estimated only a "desired speed" on tangents (e.g., Krammes et al., 1995; Fitzpatrick et al., 2000a). Several recent studies have predicted speeds on tangents (e.g., Polus, Fitzpatrick, and Fambro, 2000; Donnell et al., 2001; Gibreel et al., 2001; Schurr et al., 2002; Adolini, Minnicino, and Elefteriadou, 2004; Figueroa Medina and Tarko, 2005; Nie and Hassan, 2007), but there are still many gaps to be filled. Polus et al. (2000) identified challenges to predicting speeds on tangent sections:

> …The number of variables that influence tangent speeds are greater than those that influence speeds on curves, which makes prediction of an 85th percentile speed on tangents relatively complex. Prediction of speeds on horizontal curves may be easier than prediction of speeds on tangent sections because of the correlation of speeds to a few defined and limiting variables, such as curvature, superelevation, and the side-friction coefficients between road surface and tires. Several studies found that on curves, the radius (or degree of curve) is the most significant variable for predicting speeds at the midpoint of the curve (Krammes et al., 1995; Tarris et al., 1996; Fitzpatrick et al., 2000a). Generally, a change in these variables is controlled by designers during the planning and design process; therefore, operating speeds may be predicted and evaluated. On tangent sections of two-lane rural highways, however, the speed of vehicles is dependent on a wide array of roadway characteristics, such as the length of the tangent section, the radius of the horizontal curve before and after the section, cross-sectional elements, vertical alignment, general terrain, and available sight distance. Additionally, the operating speed on tangents largely depends on driver attitude, as well as on the acceleration and deceleration capabilities of the vehicle composition on a specific road. Therefore, it is a more complex task to predict operating speeds on tangent sections; a large database is necessary to identify any significant trends, and substantial modeling effort is required. Few studies have dealt with this issue.

Fitzpatrick and Collins (2000) concluded that "…more research is needed on the influence of various tangent lengths and grades on speeds."

Limited Research Results on Day Versus Night Speeds

Limited research results are available on differences between day and night speeds. Previous research on the effect of light conditions is much more limited than research on the effect of weather conditions on speeds (Fitzpatrick et al., 2000a). One study with an explicit objective of comparing day and night speeds was reviewed (Guzman, 1996). A short summary is included here to demonstrate that there are differences between day and night speed behavior, and future research on the topic is needed.

Guzman (1996) conducted a limited study to compare day and night speeds on horizontal curves on two-lane rural highways. Speed data were collected during the day and night under dry weather conditions at eight horizontal curves on rural two-lane highways in central Texas. The degree of curvature of the sites was as follows: three curves at 3°, two curves at 6°, two curves at 10°, and one curve at 12°. Speeds were measured at the midpoint of the approach tangent and at the curve midpoint in each lane. The terrain was flat to gently rolling. The adjacent land was a mixture of pasture and forest, and there was no lighting except for a single residence near some of the sites. Pavement markings included centerlines with supplemental retroreflective pavement markings at all sites, and edglines at four of the sites. The 6°, 10°, and 12° curves had curve warning signs with advisory speed panels, and the 12° curve also had a turn sign, large arrow sign, and chevrons.

The speed data were filtered to analyze data only for free-flowing passenger vehicles that could be tracked from the midpoint of the approach tangent to the midpoint of the curve. The initial analysis was a three-factor analysis of variance with the main effects being site, lane (inside or outside), and light condition (day or night). The number of speed observations ranged from 92 to 225 for a given condition (site, lane, and lighting). Analyses were conducted for speeds at the midpoint of the tangent, midpoint of the curve, and the speed change from the tangent to the curve. The site and lane factors and the site * lane, site * light, and sight * lane * light interactions were statistically significant at the 0.05 significance level for speeds at both the midpoint of the tangent and the midpoint of the curve. Because of the significance of these site main effect and the interactions, a site-by-site analysis was conducted in an attempt to better understand the effect of light condition.

The results indicated that day and night mean speeds differ significantly at some locations (six of 16 tangent locations and nine of 16 curve locations) but not at others. At most locations day speeds were higher (four of the six tangent locations with significant differences and seven of the nine curve locations). Day and night speed variances differed at four of the 16 tangent locations and five of the 16 curve locations. Day speed variances were higher at all four of the tangent locations with significant differences and three of the five curve locations. The mean speed reduction from tangent to curve was significantly different at five of the 16 tangent–curve pairs, and in all five cases the speed reduction was higher during the night. The significant differences in speed reductions ranged from 0.7 to 1.7 mph. Although there are some differences between day and night speeds, those differences varied among locations and no consistent effects of location (tangent or curve), curve sharpness, or lane (inside or outside) were discerned.

Many Models Are Applicable Only for Relatively Flat Terrain

Fitzpatrick et al. (2000b) noted that "…speed prediction on two-lane rural roads has been researched extensively for horizontal curves on relatively flat terrain." This implies that speed prediction on non-flat terrain has not been extensively researched.

Most Models Estimate Only a Specific Speed Percentile (e.g., 85th)

Figueroa Medina and Tarko (2005) addressed this limitation in detail, noting that "…despite a large body of past research on speeds, there is still much to learn about the factors of free-flow speeds. The existing models estimate a specific speed percentile, and they do not distinguish between the mean speed factors and the speed dispersion factors, which leads to results that are sometimes difficult to interpret. It is possible that a road with a high mean speed and low speed variability has the same 85th speed percentile as a road with a much lower mean speed but higher speed variability…Modeling the entire free-flow speed distribution suggested by some authors (Tarris et al., 1996; Fitzpatrick et al., 2003) might rectify this problem.

"The modeling approach typically used in most studies focuses on the effects of isolated or restricted alignment conditions on a specific percentile speed, typically the 85th percentile. Although the 85th percentile speed is widely used to approximate operating speeds, other percentiles have been suggested to represent a high percentage of drivers in highway design.

Further refinement is needed to develop models with the capability of predicting speeds along a roadway segment, considering the entire free-flow speed distribution and based on multiple roadway factors rather than only a fixed set of horizontal and vertical alignment combinations."

Most of the existing speed models have the following form:

$$V_i = \sum (b_k X_{ik}) + e$$

where

V_i = mean or a specific percentile speed at site i;
X_{ik} = value of the k exogenous variable at site i;
b_k = regression parameter associated with variable k; and
e = normally distributed disturbance term.

One limitation of these models is that they cannot predict any percentile other than the specific one for which they were developed. Another significant limitation is their inability to evaluate the speed variability at a site (Figueroa Medina and Tarko, 2005).

CHAPTER 6

Practitioners' Perspective

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This chapter provides a discussion that conveys the practitioners' perspective regarding the use of operating speed modeling in highway design practice. Standards or criteria are yet to be well defined related to the use of speed prediction models in the highway design process. Several improvements and advancements may be necessary to support the full understanding and application of such models.

U.S. PRACTITIONERS' PERSPECTIVE

Based on the practitioner feedback, speed models are currently used in different applications including the following.

- Determination of the 85th-percentile speed, which might be helpful to set local speed limits. In addition, many states and agencies use the speed limit plus 5 mph to establish a minimum design speed.
- In the Manual of Uniform Traffic Control Devices (MUTCD), the speed limit is a general criterion for selection and design of traffic control devices. Although the MUTCD is not generally used for geometric design, certain guidelines and dimensions in the MUTCD (e.g., criteria for marking no-passing zones) may directly influence the selection of geometric design values where there is flexibility in AASHTO's *Green Book*. In this sense, analysis of the existing operating speeds based on an engineering study or exercise of engineering judgment can influence the geometric design practice for certain features.
- Speed modeling is routinely performed in the United States in the design of roundabouts. Evaluation of the fastest path alignment and determination of an estimated operating speed through each leg of a roundabout is a standard design practice. This involves developing a theoretical design speed for each curve at the entry, circulatory roadway, and exit and by considering the rate of acceleration and deceleration between curves of different radii. The FHWA *Roundabouts: An Informational Guide* describes the methodology for performing this modeling and evaluation of predicted operating speeds.
- Speed modeling is also performed in the United States in the evaluation of critical lengths of grade and the need for climbing lanes on two-lane highways. The speed reduction of trucks to less than 10 mph below that of passenger cars is a generally accepted criterion for determining the location for beginning and ending climbing lanes, when other contributing operational conditions are present. These conditions are described in the 2004 edition of the

Green Book (p. 244). The methodology and examples of the speed modeling for trucks are described in the climbing lanes section of the *Green Book*.

- Another application of speed models is for determining the need for periodic passing lanes on two-lane roadways, such as for design of high-standard, high-volume "Super 2" highways. The capacity analysis performed for such situations also typically includes some form of modeling and an analysis of operating speeds.
- Speed analysis is commonly performed in conjunction with traffic operations capacity analyses, and in particular with traffic microsimulation. The estimated speed is an output of the capacity and level of service analyses based on speed–density relationships. In these cases the speed modeling and analysis is not used for geometric design, since higher speeds during low-volume periods will generally be the design control for the geometrics. However, such information can be used in the design of active traffic management strategies and devices for proposed facilities.
- While not yet common practice, speed modeling combined with design consistency evaluations have been used as part of safety audits and other types of in-service road safety and operational reviews.

One agency that has established operating speed as a specific design criterion is the FHWA Office of Federal Lands Highway (FLH). The FLH has developed criteria that are included in the *Project Development and Design Manual*, Chapter 9: Highway Design (http://flh.fhwa.dot.gov/resources/manuals/pddm/Chapter_09.pdf). The Section 9.3.1.13.4, p. 9-29 and Exhibit 9.3-A provides guidance on the maximum variation in operating speeds between successive curves, and between long tangents and curves, for design consistency. It is FLH standard practice to consider the variations in predicted operating speeds when developing the horizontal alignment for reconstruction projects. However, even with this guidance, most FLH projects do not receive an analysis using the IHSDM design consistency module. Although FLH has interest in using the IHSDM speed prediction models, the models are limited by the range and applicability of calibration for lower-speed facilities. FLH projects are typically on low-speed (45 mph or less) rural roads.

The current limitations of the IHSDM for low-speed (sharper radius) curves discourage many designers from performing a design consistency analysis. Despite the limitations however, many design practitioners feel that a reliable, automated and user-friendly speed prediction model would be a very helpful tool for evaluating the consistency of the design, together with their own engineering judgment. FLH recently completed a speed data collection project specifically to address application of the IHSDM for lower-speed rural roads, and this data was incorporated into the IHSDM 2010 Release. FLH should now be confident in performing an operating speed analysis for major projects where horizontal alignment alternatives are developed, and on other projects where design consistency is a specific concern. The potential application could include about one-quarter of FLH projects. It is not anticipated that FLH designers will perform an analysis of operating speed for the design of resurfacing, restoration, and rehabilitation (RRR) type projects, which represent approximately half of the agency's construction program.

Moreover, the application of operating speed modeling in the United States is complicated by national policy that establishes state responsibility for design standards and practices for most highways, except for the National Highway System. Design methodology is adopted independently in each of the 50 states, not directly by the central U.S. government.

AASHTO establishes criteria and guidelines for the states to each consider, by 2/3 ballot approval of the state DOT agencies on the content of its proposed guidelines and manuals. Most states adopt the published AASHTO guidelines, typically with supplemental standards, criteria or guidance to address each state's unique conditions. Implementation of any new methodology into highway engineering policies and professional practice in the United States takes considerable time, and requires a great deal of research, testing, collaboration, and review. The implementation of operating speed modeling and analysis using modern technologies is no exception.

Exceptions to this situation are rare and usually limited to projects of particular importance that are conducted in collaboration with workers in research centres and universities who adopt an academic approach. Therefore, the incentive to use these models must first come from road administration requirements. This is the most effective way to induce practitioners to consider the use of speed prediction models in their work when it is not explicitly recommended by guidelines.

It was also reported by highway design practitioners that speed prediction (i.e., applying the IHSDM Design Consistency Module) is rarely performed in only those few cases where

- The highway design engineer has received specific training to perform the operating speed modeling and analysis;
- The design engineer has retained an understanding of the concepts involved;
- The IHSDM modeling software has been installed on the agency's computer network for the typical design workstations;
- Specific safety concerns have been identified for an applicable project and are recognized as being related to inconsistencies in operating speeds; and
- Sufficient additional time and budget have been planned for the design engineer to perform the analysis and to incorporate and document the results.

Considering operating speeds and their consistency in U.S. design practice is largely anecdotal and based on visual observations of traffic on similar facilities to the one being designed. Typically, speeds are not explicitly predicted using methodologies such as the IHSDM and others presented in earlier chapters of this document. In general, the potential benefits of operating speed prediction modeling and analysis have not been clearly demonstrated by the research community. As a result, highway design engineers and design and project managers responsible for project delivery do not invest the additional time and money needed to routinely apply these models in practice.

Applications of operating speed modeling and analysis has primarily been reserved for a relatively small subset of non-freeway and non-RRR projects for which multiple alignment alternatives are being studied specifically in conjunction with a safety performance analysis that will be used in a formally documented process. Speed modeling is also performed for the special situations noted earlier in this chapter, including the analysis of roundabouts, truck climbing lanes, and passing lanes in conjunction with traffic operations capacity analyses where speeds are an output.

INTERNATIONAL PRACTITIONERS' PERSPECTIVE

The description of an international practitioners' perspective on operating speed prediction models is quite complex since conditions in the various countries are often very different. Questions like "Are the models being used in real life applications?" or "Is there any need from the practitioner's point of view to develop operating speed models?" do not have a single answer since the conditions in which a practitioner works are characterized not only by different rules and road design guidelines but also by different practice and professional training as well as different driving populations and behaviors.

Therefore, an exhaustive description of a practitioners' perspective requires an extensive knowledge of the specific conditions of the practitioners' activity in each country. The subject is, therefore, dealt with in general terms in this chapter, focusing on the various situations currently existing in European countries. This note is also based on brief interviews with sample of practitioners in several European countries. Not all European countries discussed in Chapter 3 provided feedback on their formal or informal use of speed models. Direct links between this chapter and Chapter 3 are not made.

The first significant distinction to be made between countries in order to correctly tackle this theme is based on the current road design regulation. There are some countries whose guidelines contain specific operating speed prediction models or the design process is based to some extent on the operating speed concept. In other countries, guidelines do not consider the operating speed concept at all. For example, the concept of operating speed has been officially used by the German Guide for the Geometric Design of Highways since 1973. In fact the guide used the operating speed as an evaluation parameter for alignment consistency. Also subsequent guides (1984 and 1995) provided different diagrams for new and existing roads to estimate the operating speed as a function of road characteristics. The guidelines of many other countries provide operating speed prediction models (for example, France, Greece, and Australia) or provide a design process that is associated with the 85th-percentile speed (for example, the United Kingdom). These guidelines usually contain specific evaluations to check the consistency of the alignment or sight distance using operating speed. Moreover, in these countries the operating speed is also used also for determining speed limits. On the contrary, there are several countries (for example, Italy, Slovenia, and Spain) whose road design guidelines do not provide any operating speed concept. In these cases, the design process generally uses a design speed that does not derive from an experimental survey but is based on theoretical considerations.

This brief description of two possible situations is complex because of the variability of the road design guideline contents but it makes it possible to better understand the different practitioner perspectives. In the first case the practitioner knows the operating speed concept well and is accustomed to using the operating speed prediction models. However, the fact that an official guideline provides a model can induce the practitioner to use this model, even though it may not be suitable for the specific situation under study. This could be the case not so much during the design process of new roads, for which the guideline was usually developed, but for safety evaluations related to minor improvements of existing roads.

In the second case, the practitioner is not usually acquainted with operating speed since this aspect is not provided for in the applicable design guidance and documentation. In this case also, road administrators who approve the projects usually are not acquainted with the operating speed concept and, therefore, do not require evaluations that involve this speed. In this context, the practitioners do not feel the need for an operating speed prediction model, simply because it is not necessary for their work. This is particularly true for the design of new roads since the actual design guidelines, even though they do not use the operating speed, make it possible nonetheless to obtain consistent alignments because they usually include different types of consistency rules.

In the case of existing roads, the lack of tools to assist the practitioner in the evaluation process is evident. In fact, many practitioners think that a reliable prediction model to estimate the speed along existing roads could be a useful tool to use in the evaluation process, alongside other tools like crash prediction models or road safety inspections. This is due to the fact that there are an increasing number of projects that concern the improvement of existing roads and not the building of new roads, especially in some European countries, with the result that the rules contained in the guidelines are usually not applicable to the existing old alignments. In this situation, the ability to estimate the speed along an existing alignment makes it possible to estimate the necessary sight distance, superelevation, possible consistency improvements, speed limits, etc. This would help the practitioner to select the right design choice for the improvement of the road. In fact, in several countries, the road administrators ask practitioners to justify the design choices of a project relating to an existing road. For example, the Italian law for the improvement of an existing road requires the practitioner to demonstrate that the project will result in an increase in the level of safety of the road but does not provide a guideline to evaluate this increase objectively.

A common criticism of practitioners of operating speed prediction models, in particular in countries such as Italy with a complex territory (old road networks, numerous intersections, many small built-up areas along rural roads, etc.), is that they do not consider characteristics such as intersections and built up areas that can greatly influence the speed and that are common along roads. This aspect limits the applicability of the prediction models.

In conclusion, practitioners do not usually feel a particular need for operating speed prediction models unless encouraged or mandated by road agency design guidelines or standards. The need for operating speed prediction models is particularly great for the evaluation of existing roadway conditions to aid in the selection/identification of effective and defensible roadway improvements. However, in this case too, it is usually the road agency that mandates the need for an evaluation process that includes speed prediction models. Clearly, the incentive to use these models must come from the client or from the regulations.

CHAPTER 7

Conclusions and Recommendations

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A review of existing operating speed profile models indicates that extensive work has been done by many researchers over decades resulting in a rich and varied body of knowledge. However, many limitations and deficiencies of the existing models were identified. This section presents a summary of existing models, and offers recommendations for future research. As summarized in Appendix A, the following conclusions can be drawn from a review of existing speed models.

- The great majority of existing models predict 85th-percentile speeds for passenger cars while relatively few studies produced truck speed models.
- Only one study (Figueroa, Medina, and Tarko, 2005) produced a model that supports a speed distribution.
- Most models predict speeds on horizontal curves while relatively few models predict speeds on tangents.
- Relatively few models consider combinations of horizontal and vertical alignment.
- Linear regression was used for most models.
- Many models contain variables related to horizontal curvature (curve radius, curve length, curve deflection angle), and quite a few contain variables related to the vertical alignment (grade, vertical curves, *K*-value), while fewer models contain variables related to tangents, horizontal–vertical combinations, cross-section elements, sight distance, and speeds (e.g., posted).
- Relatively few models include acceleration–deceleration models for speeds approaching and exiting curves.
- A majority of models assume a constant speed throughout the horizontal curve, though some do model varying speeds within a curve.

DATA COLLECTION PROCEDURES

As Hassan (2004) suggested, "…an optimum data collection procedure to capture actual drivers' speed behavior needs to be developed and agreed on. Such a procedure must not influence drivers' behavior through the introduction of perceived speed enforcement." Other recommendations related to data collection procedures include the following:

- Collect more truck and recreational vehicle speed data (field observed speeds).
- Collect data at more locations at a given site. Many studies collected data only at one location on the approach tangent and one location on the horizontal curve (usually the curve midpoint). More data collection locations, from the approach tangent through the curve and departure tangent would improve modeling capabilities. In selecting data collection locations, consider that minimum truck speeds may not occur at the same locations as minimum passenger car speeds, and consider performance implications of the vertical alignment gradient.
- Strive for large sample sizes (number of sites and observations per site). As concluded by Fitzpatrick et al. (2003), the reason for not being able to estimate some variables that seem to influence speeds to a good statistical accuracy is most likely due to the limited number of sites available for analysis.
- Collect data for a greater range of conditions (e.g., wider range of curve radii) .
- Collect data to determine the effect of time of data collection (day versus night), weather conditions, and non-flat terrain.

VALIDATING DRIVER BEHAVIOR ASSUMPTIONS

More research is recommended to validate driver behavior assumptions, including the following:

- Investigate acceleration and deceleration behavior. Many models assume a constant speed throughout horizontal curves, while acknowledging that this is not actually the case.
- Assess the impact of vertical curves, as well as horizontal–vertical combinations (e.g., to investigate the assumption that behavior on vertical alignments or vertical–horizontal combinations is the same as on horizontal alignments).
- Investigate the assumption that the speed relationships for all crest vertical curve types and all sag vertical curve types are similar (Fitzpatrick et al., 2000a).

As recommended by Fitzpatrick et al. (2003) "…conduct research that emphasizes drivers' speed choice behaviors…There are perhaps three types of drivers in terms of their speed choices: (*a*) conservative drivers who always try to stay below the posted speed limit, (*b*) moderate drivers, who constitute the majority of the drivers, who try not to exceed the speed limit to an unreasonable degree, and (*c*) aggressive drivers who use the posted speed limit as the lower bound and constantly look for opportunities to drive at higher speeds. This kind of research recognizes the importance of human factors in determining driving speeds and the heterogeneity of the driver population."

ESTIMATING SPEED CHANGES BETWEEN GEOMETRIC ELEMENTS

There is a need to continue the efforts of McFadden and Elefteriadou (2000), Misaghi and Hassan (2005), and Nie and Hassan (2007) to address the deficiencies in measuring speed reductions between tangents and curves in terms of $\Delta V85$ (which assumes that speed distribution on successive elements is the same, and is calculated by simple subtraction of 85th-percentile speeds on the tangent and curve). Also, as suggested by McFadden and Elefteriadou (2000), "…research that identifies where drivers reach the maximum speed on the approach tangent and minimum speed on the horizontal curve would be valuable."

TRUCK SPEED MODELS

Further research should be conducted on estimating operating speeds of trucks and recreational vehicles for different horizontal and vertical curves (Fitzpatrick et al., 2000a). Donnell et al. (2001) suggested that additional research is required in developing truck operating speed models.

- Truck speed data should be collected from field sites with varying combinations of horizontal and vertical curves, and the predictive capabilities of TWOPAS for such sites should be assessed. Future research should include sag and crest vertical curves (limited sight distance and nonlimited sight distance), in combination with a good range of horizontal alignments.
- Their study did not consider the differences in operating speeds caused by the varying performance capabilities of trucks (mostly caused by differences in the weight-to-horsepower ratio). These differences may be significant, especially for steep grades. Future research should consider different vehicle performance characteristics.

Adolini-Minnicino and Elefteriadou (2004) also concurred that: "…ideally, operating speed models for trucks would be based on field data from a variety of horizontal and vertical geometric conditions." They suggested that "…if this type of field data is not available, the models can be based on simulated data if the simulation model has been validated for means and speed variances." Future use of TWOPAS in the development of operating speed models is recommended if TWOPAS can be calibrated to acceptably predict operating speed distributions (Adolini-Minnicino and Elefteriadou, 2004).

MODELING TECHNIQUES

Advances in modeling approaches are needed to overcome the limitations of linear regression modeling, which include a tendency to overstate or understate the influence of geometric elements, a flawed assumption of data independence, and inflation of R^2 due to the use of aggregate data. Recommendations in this area include

- Strive to rely on disaggregate data whenever possible;
- Control for speed and speed variance so that their effects are not confused; and
- Explore further development of panel data models— Figueroa, Medina, and Tarko (2005) discussed advantages of their OLS-PD model, including
  - Predicting any user-specified percentile,
  - Involving more design variables than traditional OLS models,
  - Separating the impacts on mean speed from the impacts on speed dispersion,

- By having a higher number of observations than a typical cross-sectional data set, a panel has more degrees of freedom, thereby reducing collinearity between the explanatory variables and improving the efficiency of the parameter estimates, and
- The OLS-PD model can be further improved by adding site-specific and percentile-specific random effects to avoid bias in estimating the model parameters caused by unknown factors not incorporated in the regression model.

Hassan (2004) also noted potential advantages of the panel data approach: "…the findings of Misaghi (2003) that operating speed is not strongly correlated to alignment features need to be further verified once the optimum data collection procedure has been developed. If this finding is confirmed using a larger database, alternative approaches to the simple regression analysis should be developed to predict operating speed on the different features of the highway alignment. An alternative technique has been suggested already based on panel analysis and was reported to be able to capture the effect of individual driver's speed choice (Tarris et al., 1996):

- Continue to explore the use of ANNs to develop speed models, as investigated by McFadden et al. (2001) and
- Explore development of multilevel regression models, which show promise in increasing the accuracy and precision of speed differential estimates, possibly with less data, as reported by Park et al. (2010).

Econometric, systems modeling approaches should be explored when more than one operational speed measure is of interest or when potential endogeneity issues exist with variables on the right-hand side of the speed prediction model. Systems modeling approaches applicable to speed versus road geometry relationships include seemingly unrelated regression (Porter et al., 2007; Porter and Mason, 2008) and simultaneous equation models (Shankar and Mannering, 1998; Porter, 2007; Himes and Donnell, 2010). The systems models, sometimes referred to as full-information models, provide greater insights to the underlying speed relationships and improve efficiency of parameter estimates (i.e., reduce standard errors). They have also been used to address some complex modeling and interpretation issues associated with the inclusion or exclusion of posted speed from speed prediction models (Himes et al., 2010).

MODELING NIGHTTIME SPEEDS

Operating speed models and design consistency evaluations have safety implications. This synthesis has demonstrated that relatively little is known about speed behavior at night, even though limited research has uncovered possible differences in speed behavior when compared to daytime conditions (Guzman, 1996).

Night roadway travel is generally regarded as riskier than day travel. This conclusion is usually supported by combining data from several sources that include the proportion of severe crashes at night with the proportion of travel at night. For example, Mace and Porter (2004) estimated relative risk of night driving in their FHWA research report on fixed roadway lighting using data from the NHTSA and the National Safety Council. They estimated that around 45% of traffic fatalities occur at night even though the number of vehicle miles driven at night represents about 14% of the total. They further demonstrated the severity of the nighttime accident problem by estimating a nighttime traffic death rate of 4.63 deaths per 100 million vehicle miles, 4.4 times higher than in the day. Mace and Porter (2004) reported that around 56% of nighttime traffic fatalities occurred on rural roads while only 40% of all vehicle mileage occurred on rural roads. Finally, they concluded that the rural nighttime death rate has consistently been about three times that found in rural daytime settings and about two-and-a-half times that of urban nighttime settings.

The safety numbers reported by Mace and Porter (2004) indicate that ideas such as operating speed–design consistency may be a more important consideration at night than during the day. However, almost all previous speed modeling work only analyzed night speeds if enough data were available, i.e., any nighttime analysis was treated as secondary in the studies – the primary focus was dry, daytime conditions. As a result, speed prediction capabilities at night, particularly as a function of roadway geometry and traffic control, do not exist. Future speed modeling work on all roadway types should include study designs where nighttime speed models can be estimated and interpreted at a much greater level of detail than in the current body of published speed modeling literature.

EXPANDING THE APPLICABILITY OF MODELS

Research in several areas is recommended to expand the applicability of models.

Models That Predict More Than a Specific Speed Percentile (e.g., 85th)

Desirable characteristics include the following.

- Distinguish mean speed factors from speed dispersion factors.
- Estimate the entire range of speed variability at a site (modeling the entire free-flow speed distribution might rectify this problem). As recommended by Fitzpatrick et al. (2003), "…evaluate the effects of considering the entire speed distribution, instead of focusing on a particular percentile speed."
- Base the speed models on multiple roadway factors rather than only a fixed set of horizontal and vertical combinations.

Tangent Speeds

Prediction of an 85th-percentile speed on tangents is relatively complex due to the great number of variables that influence tangent speeds (Polus et al., 2000). More research is needed to predict tangent speeds.

- Fitzpatrick and Collins (2000) stated "…additional insight into the influences of speeds on tangent sections of various lengths and grades is needed. This proposed research would greatly enhance the effectiveness of any speed-profile model because it may validate or modify the assumptions currently being made."
- Fitzpatrick et al. (2000b) concluded that "…further research is needed on the ability of alignment indices to estimate desired speeds of motorists on long tangents of two-lane rural

highways. Gaps in the existing database included roads with posted speeds greater than 88.5 km/h (55 mph) and alignment indices near the maximum values of those in this study."

- Polus et al. (2000) noted the need for additional research related to tangent speeds: "…the models…were preliminary, and they clearly need additional data. Further research is also suggested on the impact of some secondary variables, such as the cross-section elements (lane width and roadside characteristics); the direction of the curves, and the longitudinal slope on the 85th percentile speed on two-lane rural highways."

Impact of Horizontal–Vertical Alignment Combinations

While some models consider horizontal alignment–vertical alignment combinations (e.g., IHSDM, Adolini-Minnicino and Elefteriadou, 2004), most do not. For example, and Collins (2000) noted that "…the acceleration and deceleration models developed were exclusively related to the impact of the horizontal curve. It is recommended that a similar effort be undertaken to assess the impact of limited-sight-distance vertical curves, as well as horizontal–vertical combinations on acceleration and deceleration profiles."

Effect of Various Geometric Elements and Configurations on Speed

More research is needed on the effect of various geometric elements and configurations on speed, including the following:

- Spiral transition curves: Passetti and Fambro (1999) made recommendations to conduct further research comparing spiralled transition curves to similar circular curves to study the side friction demands of both design alternatives, and to determine if vehicle deceleration and acceleration are affected by the presence of spiral transitions. Such research would involve collecting data at many points throughout the spiral transitions and comparing the data with similar circular curves.
- Effect of presence of intersections and access points (driveways).
- Effect of change in number of lanes (e.g., turn lanes).

Extend Models for Applicability to a Wider Range of Speeds

Most models appear to apply mainly to higher speed highways (posted speeds greater than or equal to 50 mph). Research is needed to develop or extend models for lower speed two-lane rural roads. One study in this area is FHWA's effort that collected additional data on lower-speed roadways and calibrated the speed-prediction model in the 2010 release of the IHSDM Design Consistency Module.

Address the Issue of Model Range of Applicability Versus Model Accuracy

As stated by Hassan (2004), "…the optimum size of an area to be covered by a prediction model needs to be estimated. The trade-offs between developing a more general, but less accurate, model on the basis of the data from a large area and a more specific, but more accurate, model covering a smaller area must be considered."

APPENDIX A

North America

Operating Speed Studies for Two-Lane, Rural Highways

TABLE A-1 A Summary of Model Details

	Leisch and Leisch, 1977	Lamm et al., 1987–1990	Krammes et al., 1995	Morrall and Talarico 1994	Islam and Seneviratne, 1994	Voigt and Krammes, 1996	Passetti and Fambro, 1999	Fambro et al., 2000	McFadden and Elefteriadou, 2000	Fitzpatrick and Collins, 2000	Fitzpatrick et al., 2000	Polus, Fitzpatrick, and Fambro, 2000	McFadden et. al., 2001	Donnell et al., 2001	Gibreel et al., 2001	Jessen et al., 2001	Schurr et al., 2002	IHSDM, 2003	Adolini-Minnicino and Elefteriadou, 2004	Figueroa Medina and Tarko, 2005	Schurr, Spargo, Huff, and Pesti, 2005	Misaghi and Hassan, 2005	Nie and Hassan, 2007
Vehicle Types																							
Passenger cars	X	X	X	X	X	X	X	X	X	X	X	X	X		X	X	X	X		X	X	X[3]	X
Trucks	X	X[4]												X					X		X		
Speeds																							
85th percentile		X	X	X	X	X	X	X	X[1]	X	X	X	X	X	X	X	X	X		X[2]		X	X
Mean	X															X	X		X	X			
95th percentile																X	X			X[2]	X		
Speed distribution																				X			
Speed Reduction From Tangent to Curve																							
Delta *V*85		X	X							X	X							X					X
Delta 85*V*																						X	X
85MSR									X														X
85MSI																							X

NOTES:

[1] Only 85MSR (no speed profile)

[2] Mean speed predicted; all percentiles can be derived

[3] Passenger car model also applies to light trucks (20% of observations)

[4] Model predicts *V*85 for all vehicle types (passenger cars, pickups, vans, trucks)

(*continued on next page)*

TABLE A-1 (*continued*) **A Summary of Model Details**

	Leisch and Leisch, 1977	Lamm et al., 1987–1990	Krammes et al., 1995	Morrall and Talarico 1994	Islam and Seneviratne 1994	Voigt and Krammes 1996	Passetti and Fambro 1999	Fambro et al. 2000	McFadden and Elefteriadou 2000	Fitzpatrick and Collins 2000	Fitzpatrick et al. 2000	Polus, Fitzpatrick and Fambro 2000	McFadden et. al. 2001	Donnell et al. 2001	Gibreel et al. 2001	Jessen et al. 2001	Schurr et al. 2002	IHSDM, 2003	Adolini-Minnicino and Elefteriadou 2004	Figueroa Medina and Tarko 2005	Schurr, Spargo, Huff and Pesti 2005	Misaghi and Hassan 2005	Nie and Hassan 2007
Design Elements on Which Speeds Are Predicted																							
Horizontal curves	X	X	X	X	X	X	X		X[5]	X	X		X	X	X		X	X	X	X	X[7]	X	X
Horizontal tangents	X?		D						X[5]	D	D	X		X	X		X	D	X	X	X[7]	D	X
Vertical curves	X							X		X	X				X	X[6]		X	X				
Vertical grades																X							
Combination of horizontal & vertical	X									X	X				X			X	X				
Multiple locations on tangent and curve			X[10]							X[10]	X[10]			X[9]	X[8]			X[10]	X[9]				X[8]
Model Type																							
OLS regression	X	X	X	X	X	X	X	X	X	X	X	X		X	X	X	X	X	X		X	X	X
OLS-PD																				X			
ANN													X										

NOTES: D = desired speed on tangent (i.e., speed is not predicted by model)
[5] For maximum speed reduction (85MSR)
[6] Crest vertical curves only
[7] Speed predicted at a "distance from stop line"
[8] Speed predicted on approach and departure tangents; begin/middle/end curves
[9] At PC-200/150/100/50 and PC; curve QP, MP, 3QP; PT, PT+50/100/150/200
[10] Continuous operating speed profile

(*continued on next page)*

TABLE A-1 (*continued*) **A Summary of Model Details**

	Leisch and Leisch 1977	Lamm et. al 1987-90	Krammes et al. 1995	Morrall and Talarico 1994	Islam and Seneviratne 1994	Voigt and Krammes 1996	Passetti and Fambro 1999	Fambro et al. 2000	McFadden and Elefteriadou 2000	Fitzpatrick and Collins 2000	Fitzpatrick et al. 2000	Polus, Fitzpatrick and Fambro 2000	McFadden et. al. 2001	Donnell et al. 2001	Gibreel et al. 2001	Jessen et al. 2001	Schurr et al. 2002	IHSDM, 2003	Adolini-Minnicino and Elefteriadou 2004	Figueroa Medina and Tarko 2005	Schurr, Spargo, Huff and Pesti 2005	Misaghi and Hassan 2005	Nie and Hassan 2007
VARIABLES IN MODEL – Horizontal Alignment																							
Curve radius/degree of curve		X	X	X	X	X	X		X	X	X	X[12]	X	X	X			X	X	X		X	X
Horizontal curve length		X[15]	X			X							X				X		X				X[15]
Curve deflection angle			X			X							X		X		X		X			X[14]	X[16]
Spiral curves		X[15]					X[18]																X[15]
Curve direction																						X	
Preceding and succeeding curves												X											
Tangent length			X[17]						X	X[17]	X[17]	X		X				X[17]	X				X
Horizontal/vertical combination										X[11]	X[11]				X			X[11]	X[13]				

[11] Used to select alignment condition (AC)
[12] Preceding and succeeding curves, for tangent speed
[13] “Alignment Categories” include horizontal/vertical combinations
[14] For estimating speed reduction (Delta 85*V*)
[15] Used for calculating curvature change rate (CCR)
[16] For estimating speed reduction (85MSR)
[17] For estimating acceleration/deceleration rates
[18] Examined the effect of spirals on curve speeds, but spirals are not a model variable

(*continued on next page)*

TABLE A-1 (*continued*) A Summary of Model Details

	Leisch and Leisch, 1977	Lamm et al., 1987–1990	Krammes et al., 1995	Morrall and Talarico, 1994	Islam and Seneviratne, 1994	Voigt and Krammes, 1996	Passetti and Fambro, 1999	Fambro et al., 2000	McFadden and Elefteriadou, 2000	Fitzpatrick and Collins, 2000	Fitzpatrick et al., 2000	Polus, Fitzpatrick, and Fambro, 2000	McFadden et. al., 2001	Donnell et al., 2001	Gibreel et al., 2001	Jessen et al., 2001	Schurr et al., 2002	IHSDM, 2003	Adolini-Minnicino and Elefteriadou, 2004	Figueroa, Medina, and Tarko, 2005	Schurr, Spargo, Huff, and Pesti, 2005	Misaghi and Hassan, 2005	Nie and Hassan, 2007
VARIABLES IN MODEL – Vertical Alignment																							
Vertical grade	X									X[19]	X[19]			X	X	X	X	X[19]	X	X		X	
Vertical curves										X[19]	X[19]				X			X[19]	X				
K-value										X	X				X				X				
VARIABLES IN MODEL – Cross Section																							
Superlevation/ cross slope						X									X				X	X			
Lane width		X[22]																		X			
Shoulder width		X																		X		X	
Roadside clear zone																				X			
VARIABLES IN MODEL – Sight Distance																							
Horizontal (limited)	X[21]																			X[20]			
Horizontal (nonlim.)	X[21]																			X[20]			
Vertical (limited)	X[21]									X[19]	X[19]									X[20]			
Vertical (nonlim.)	X[21]									X[19]	X[19]									X[20]			

[19] To select alignment condition (AC) and/or for TWOPAS performance-limited speed predictions.
[20] Type of sight distance is not specified.
[21] Acceleration rates departing a horizontal curve are a function of available sight distance.
[22] Different *V*85 models were developed for different lane widths.

(*continued on next page*)

TABLE A-1 (*continued*) A Summary of Model Details

	Leisch and Leisch, 1977	Lamm et al, 1987–1990	Krammes et al., 1995	Morrall and Talarico, 1994	Islam and Seneviratne, 1994	Voigt and Krammes, 1996	Passetti and Fambro, 1999	Fambro et al., 2000	McFadden and Elefteriadou, 2000	Fitzpatrick and Collins, 2000	Fitzpatrick et al., 2000	Polus, Fitzpatrick, and Fambro, 2000	McFadden et. al., 2001	Donnell et al., 2001	Gibreel et al., 2001	Jessen et al., 2001	Schurr et al., 2002	IHSDM, 2003	Adolini-Minnicino and Elefteriadou, 2004	Figueroa, Medina, and Tarko, 2005	Schurr, Spargo, Huff, and Pesti, 2005	Misaghi and Hassan, 2005	Nie and Hassan, 2007
VARIABLES IN MODEL – Speeds																							
Inferred design speed of curve								X[23]															
Posted speed		X[30]														X	X			X[24]			
Approach tangent	X		X						X													X[25]	
VARIABLES IN MODEL – Other																							
AADT		X														X	X						
Distance to stop																					X		
Percent trucks																				X			
Residential drives/mi																				X			
Nearby intersection																				X			
Driveway on curve flag																						X	

[23] For vertical curves.
[24] Flag for 50 or 55 mph posted speed.
[25] For estimating speed reduction (Delta*V*85).
[30] Posted recommended speed on curves or curved section.

(*continued on next page)*

TABLE A-1 (*continued*) A Summary of Model Details

	Leisch and Leisch, 1977	Lamm et al., 1987–1990	Krammes et al., 1995	Morrall and Talarico, 1994	Islam and Seneviratne, 1994	Voigt and Krammes, 1996	Passetti and Fambro, 1999	Fambro et al., 2000	McFadden and Elefteriadou, 2000	Fitzpatrick and Collins, 2000	Fitzpatrick et al., 2000	Polus, Fitzpatrick, and Fambro ,2000	McFadden et. al., 2001	Donnell et al., 2001	Gibreel et al., 2001	Jessen et al., 2001	Schurr et al., 2002	IHSDM, 2003	Adolini-Minnicino and Elefteriadou, 2004	Figueroa, Medina, and Tarko, 2005	Schurr, Spargo, Huff, and Pesti, 2005	Misaghi and Hassan, 2005	Nie and Hassan, 2007
ACCELERATION / DECELERATION MODEL																							
Model for acceleration out of curve and deceleration into curve	X		X							X	X							X		X			
HORIZONTAL CURVE SPEED ASSUMPTION																							
Constant speed	X?	X	X	X		X	X	X[26]		X	X	n/a	X			n/a	X	X	X[28]		n/a	X	
Varying speed					X[29]				X	X[27]	X[27]	n/a		X	X[29]	n/a		X[27]	X[28]	X	n/a		X

[26] For vertical curves.
[27] TWOPAS performance-limited speeds can vary within a curve.
[28] Curve speed predicted at curve midpoint only in some models; at curve quarter points in others.
[29] Models for $V85$ at PC, middle of curve (MC) and PT.

TABLE A-2 Study Summary: Number of Sites and Observations per Site

Study	Country	Number of Sites	Number of Observations/Site
Leisch and Leisch. 1977	USA	?	?
Lamm et al., 1987–1990	USA	261 curves	N/A
Krammes et al., 1995	USA	138 circular curves; 78 approach tangents	50-100[1]
Morrall and Talarico, 1994	Canada	9[1]	N/A[1]
Islam and Seneviratne, 1994	USA	8 curves	125
Voigt and Krammes, 1996	USA	138 circular curves; 78 approach tangents	At least 100 per site
Passetti and Fambro, 1999	USA	51	At least 100 per site
Fambro et al., 2000	USA	36	3,500 paired speeds (control site and crest vertical curve); minimum 4 hours or 100 vehicles/site
McFadden and Elefteriadou, 2000	USA	21	At least 75 per site
Fitzpatrick and Collins, 2000	USA	See Fitzpatrick et al., 2000	See Fitzpatrick et al. 2000
Fitzpatrick et al., 2000	USA	176 sites (103 for model development; 73 for validation); 21 curves for accel. / decel. models	At least 100 per site[1]
Polus, Fitzpatrick, and Fambro, 2000	USA	162 tangent sections	At least 100 per site
McFadden et. al., 2001	USA	See Krammes et al., 1995	
Donnell et al., 2001	USA	13	At least 100 per site[1]
Gibreel et al., 2001	Canada	38[1]	1 hour[1]
Jessen et al., 2001	USA	70	At least 275 per site
Schurr et al., 2002	USA	40	?
IHSDM, 2003	USA	See Fitzpatrick et al., 2000	See Fitzpatrick et al. 2000
Adolini-Minnicino and Elefteriadou, 2004	USA	78 (11 with observations at 13 locations through tangent and curve; 67 with observations at tangent midpoint and curve midpoint only)	
Figueroa Medina and Tarko, 2005	USA	158 “spots” (including 85 on tangent/flat curves; 14 on sharp curves)	360 per “spot” on average; 100 per spot minimum
Schurr, Spargo, Huff, and Pesti, 2005	USA	15 (3 tangents, 12 curves)	?
Misaghi and Hassan, 2005	Canada	20 curves x 2 directions = 40 sites	At least 100 per site
Nie and Hassan, 2007	Canada	10 curves and approach tangents	25 per site, on average

[1] From Hassan, 2004.

APPENDIX B

North America

List of Two-Lane, Rural Highway Studies

VEHICLE TYPE	
Passenger car speeds	• All, except Donnell et al., 2001; Adolini-Minnicino and Elefteriadou, 2004
Truck speeds	• Leisch and Leisch, 1977 • Lamm et al., 1987–1990 • Donnell et al., 2001 • Adolini-Minnicino and Elefteriadou, 2004 • Schurr, Spargo, Huff, and Pesti, 2005

SPEEDS PREDICTED	
85th percentile	• All, except Leisch and Leisch, 1977; Adolini-Minnicino and Elefteriadou, 2004; Schurr, Spargo, Huff, and Pesti, 2005
Mean	• Leisch and Leisch, 1977 • Jessen et al., 2001 • Schurr et al., 2002 • Adolini-Minnicino and Elefteriadou, 2004 • Figueroa, Medina, and Tarko, 2005
95th percentile	• Jessen et al., 2001 • Schurr et al., 2002 • Figueroa, Medina, and Tarko, 2005 • Schurr, Spargo, Huff, and Pesti, 2005 • Leisch and Leisch, 1977 • Fambro et al., 2000 • Fitzpatrick et al., 2000 • IHSDM, 2003
Speed distribution	• Figueroa Medina and Tarko, 2005

DESIGN ELEMENTS ON WHICH SPEEDS ARE PREDICTED	
Horizontal curves	• All, except Fambro et al., 2000; Polus, Fitzpatrick, and Fambro, 2000; and Jessen et al., 2001
Tangents (predicted)	• Leisch and Leisch, 1977 • McFadden and Elefteriadou, 2000 • Polus, Fitzpatrick, and Fambro, 2000 • Donnell et al., 2001 • Gibreel et al., 2001 • Schurr et al., 2002 • Adolini-Minnicino and Elefteriadou, 2004 • Figueroa, Medina, and Tarko, 2005 • Schurr, Spargo, Huff and Pesti, 2005 • Nie and Hassan, 2007

Tangents ("desired" speed)	• Krammes et al., 1995 • Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • IHSDM, 2003 • Misaghi and Hassan, 2005
Vertical curves	• Leisch and Leisch, 1977 • Fambro et al., 2000 • Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • Gibreel et al., 2001 • Jessen et al., 2001 • IHSDM, 2003 • Adolini-Minnicino and Elefteriadou, 2004
Vertical grades	• Jessen et al., 2001
Combinations of horizontal and vertical alignments	• Leisch and Leisch, 1977 • Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • Gibreel et al., 2001 • IHSDM, 2003 • Adolini-Minnicino and Elefteriadou, 2004
Multiple locations on approach tangent and within curve	• Krammes et al., 1995 • Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • Donnell et al., 2001 • Gibreel et al., 2001 • IHSDM, 2003 • Adolini-Minnicino and Elefteriadou, 2004 • Nie and Hassan, 2007

MODEL TYPE	
Linear regression	• All, except McFadden et al., 2001; Figueroa, Medina, and Tarko, 2005
Artificial neural network	• McFadden et al., 2001
OLS-panel data	• Figueroa, Medina, and Tarko, 2005

VARIABLES IN MODEL	
Horizontal curve radius/degree of curve	• Lamm et al., 1987–1990 • Morrall and Talarico, 1994 • Islam and Seneviratne, 1994 • Krammes et al .,1995 • Voigt and Krammes, 1996 • Passetti and Fambro, 1999 • McFadden and Elefteriadou, 2000 • Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • Polus, Fitzpatrick, and Fambro, 2000 • McFadden et al., 2001

	• Donnell et al., 2001 • Gibreel et al., 2001 • IHSDM, 2003 • Adolini-Minnicino and Elefteriadou, 2004 • Figueroa, Medina, and Tarko, 2005 • Misaghi and Hassan, 2005 • Nie and Hassan, 2007
Horizontal curve length	• Lamm et al., 1987–1990 • Krammes et al., 1995 • Voigt and Krammes, 1996 • McFadden et al., 2001 • Schurr et al., 2002 • Adolini-Minnicino and Elefteriadou, 2004 • Nie and Hassan, 2007
Horizontal curve deflection angle	• Krammes et al., 1995 • Voigt and Krammes, 1996 • McFadden et al., 2001 • Gibreel et al., 2001 • Schurr et al., 2002 • Adolini-Minnicino and Elefteriadou, 2004 • Misaghi and Hassan, 2005 • Nie and Hassan, 2007
Spiral transition curves	• Lamm et al., 1987–1990 • Passetti and Fambro. 1999 • Nie and Hassan. 2007
Tangent length	• Krammes et al., 1995 • McFadden and Elefteriadou, 2000 • Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • Polus, Fitzpatrick, and Fambro, 2000 • Donnell et al., 2001 • IHSDM, 2003 • Adolini-Minnicino and Elefteriadou, 2004 • Nie and Hassan, 2007
Horizontal/vertical combinations	• Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • Gibreel et al., 2001 • IHSDM, 2003 • Adolini-Minnicino and Elefteriadou, 2004
Vertical grade	• Leisch and Leisch, 1977 • Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • Donnell et al., 2001 • Gibreel et al., 2001 • Jessen et al., 2001 • Schurr et al., 2002 • IHSDM, 2003

	• Adolini-Minnicino and Elefteriadou, 2004 • Figueroa, Medina, and Tarko, 2005 • Misaghi and Hassan, 2005
Vertical curve	• Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • Gibreel et al., 2001 • IHSDM Design Consistency Module, 2003 • Adolini-Minnicino and Elefteriadou, 2004
K-value	• Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • Gibreel et al., 2001 • Adolini-Minnicino and Elefteriadou, 2004
Superelevation/cross slope	• Voigt and Krammes, 1996 • Gibreel et al., 2001 • Adolini-Minnicino and Elefteriadou, 2004 • Figueroa, Medina, and Tarko, 2005
Lane width	• Lamm et al., 1987-90 • Figueroa, Medina, and Tarko, 2005
Shoulder width	• Lamm et al., 1987–1990 • Figueroa, Medina, and Tarko, 2005 • Misaghi and Hassan, 2005
Sight distance	• Leisch and Leisch, 1977 • Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • Figueroa, Medina, and Tarko, 2005
Inferred design speed of vertical curve	• Fambro et al., 2000
Posted speed	• Lamm et al., 1987–1990 • Jessen et al., 2001 • Schurr et al., 2002 • Figueroa, Medina, and Tarko, 2005
Speed on approach tangent	• Leisch and Leisch, 1977 • Krammes et al., 1995 • McFadden and Elefteriadou, 2000 • Misaghi and Hassan, 2005
AADT	• Lamm et al., 1987–1990 • Jessen et al., 2001 • Schurr et al., 2002

ACCELERATION/DECELERATION MODEL	
Acceleration/deceleration model	• Leisch and Leisch, 1977 • Krammes et al., 1995 • Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • IHSDM, 2003 • Figueroa, Medina, and Tarko, 2005

HORIZONTAL CURVE SPEED ASSUMPTION	
Constant speed throughout curve	• Leisch and Leisch, 1977 • Lamm et al., 1987–1990 • Morrall and Talarico, 1994 • Krammes et al., 1995 • Voigt and Krammes, 1996 • Passetti and Fambro, 1999 • Fambro et al., 2000 • Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • McFadden et al., 2001 • Schurr et al., 2002 • IHSDM, 2003 • Adolini-Minnicino and Elefteriadou, 2004 • Misaghi and Hassan, 2005
Varying speed within curve	• Islam and Seneviratne, 1994 • McFadden and Elefteriadou, 2000 • Fitzpatrick and Collins, 2000 • Fitzpatrick et al., 2000 • Donnell et al., 2001 • Gibreel et al., 2001 • IHSDM, 2003 • Adolini-Minnicino and Elefteriadou, 2004 • Figueroa Medina and Tarko, 2005 • Nie and Hassan, 2007

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