

Operating Speed Prediction on Crest Vertical Curves of Rural Two-Lane Highways in Nebraska

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ABSTRACT

Since the repeal of the National Maximum Speed Limit by the United States Congress, individual states have been modifying existing speed limits on state and federal highways. Increases in speed limits have caused the inferred design speeds of many crest vertical curves along two-lane rural highways to be less than the posted and operating speeds according to accepted national guidelines. Past research has found that rate of curvature and design speed can be used in predicting operating speeds on crest vertical curves. This study includes crest vertical curve, roadway, traffic, and speed characteristics data to determine the features that influence the speeds on crest vertical curves.

Speed data were collected on 70 crest vertical curves across Nebraska. Multiple linear regression was used to determine the crest vertical curve, roadway, traffic, and speed characteristics affecting speeds on crest vertical curvature. The posted speed of the highway was found to have the most influence on the operating speed. The design speed of the vertical curves failed to be a significant factor.

OBJECTIVE

The objective of this research was to clarify the relationship between design, operating, and posted speed along crest vertical curvature on horizontal tangents in Nebraska. Some highways in the state currently have posted speed limits greater than the speed chosen to design their three-dimensional features when constructed. The Nebraska Department of Roads is currently authorized to set speed limits on the state's highways since the repeal of the National Maximum Speed Limit in 1996 and initiated this research to provide guidance for the establishment of appropriate speed limits and design speeds. Proper crest curvature design is deemed critical for safe vehicle operation since the roadway effectively blocks the view of the

driver when approaching a crest and due to higher accident rates at crests when compared to level tangent sections of roadway (1). This research investigates crest vertical curve, roadway, traffic, and speed characteristics in order to determine which factors affect operating speeds.

BACKGROUND ON SPEED AND CREST VERTICAL CURVATURE

Crest vertical curvature provides a transition between tangent profile segments along a roadway. Desirably, drivers should be able to safely and comfortably traverse crests without significantly adjusting their desired speed from that on level sections. According to the nation's leading guide for roadway design (2), the posted speed should represent the 85th-percentile speed of the vehicles using the facility and the alignments should be designed to support the 95th-percentile speed. In theory, the design speed should be higher than the posted speed, which should be greater than or equal to the 85th-percentile operating speed. Practically, these speeds should be at least comparable in magnitude. This will promote vehicle speed uniformity, which has the benefit of reducing the likelihood of vehicle collisions (3). Accurately predicting the 85th- and 95th-percentile speed of a vehicle on a crest allows the engineer to design the vertical features of the highway to better accommodate the driver. Theoretically, the design speed used to establish the characteristics of a crest curve should desirably accommodate the 95th-percentile observed speed and minimally meet the 85th-percentile speed. The crest curve geometrics would be expected to produce an operating speed that would be consistent with desired speeds along roadway tangent segments. A consistent road design should not inhibit the ability of drivers to safely control their vehicles (2) and should ensure that "most drivers would be able to operate safely at their desired speed along the entire alignment (4)."

For two-lane rural highways, design speed determines the length of vertical curves, which in turn have a direct bearing on the quality and cost of the designed facility (1). A study

by Messer resulted in the conclusion that sight distance is the primary factor affecting driver expectancy when looking at the vertical alignment of the road (5). Fambro et al. determined that operating speeds are well above the inferred design speeds on crest vertical curves (6). The following equation was developed from 42 vertical crest curves in three states and found to be a fairly good predictor of 85th-percentile speeds for two-lane rural roads with shoulders less than 1.8 m (6 ft) (6).

$$V_{85} = 72.74 + 0.47V_D \quad (1)$$

where,

$$\begin{aligned} V_{85} &= 85^{\text{th}}\text{-percentile speed (km/h); and} \\ V_D &= \text{inferred design speed (km/h).} \end{aligned}$$

However, a statistically significant relationship between inferred design speed and operating speed could not be established for roadways with shoulders greater than 1.8 m (6 ft).

Also, operating speeds on crest vertical curves with limited sight distance were found to be a function of the rate of curvature in a recent study by Fitzpatrick et al. (7). Sites for the study were located in Washington and Texas. To improve the effort to find a good prediction equation, vertical curve sites were separated into two groups; those with limited sight distance and non-limited sight distance. Limited sight distance for the study was defined as having adequate distance for a comfortable stop on wet pavement after perceiving an object in the vehicle's path when the initial speed of a passenger car is 77 km/h (48 mph) (7). The following equation was developed to predict operating speeds on limited-sight-distance vertical curves on horizontal tangents:

$$V_{85} = 111.07 - \frac{175.98}{K} \quad (2)$$

where,

$$\begin{aligned} V_{85} &= 85^{\text{th}}\text{-percentile speed (km/h); and} \\ K &= \text{Rate of vertical curvature (m per \% change in grade).} \end{aligned}$$

Another equation from the same study was developed for sites where data was taken before the repeal of the national speed limit of 88.5 km/h (55 mph). The following prediction equation resulted:

$$V_{85} = 105.08 - \frac{149.69}{K} \quad (3)$$

where,

$$\begin{aligned} V_{85} &= 85^{\text{th}}\text{-percentile speed (km/h); and} \\ K &= \text{rate of vertical curvature (m per \% change in grade).} \end{aligned}$$

Since Equation 2 would yield a higher operating speed than Equation 3 given the same K-value, one could conclude that the posted speed may have an effect upon operating speeds. Equations 1, 2, and 3 are evaluated using the site data collected from Nebraska in this study. Results of the evaluation of the equations are discussed later.

STOPPING SIGHT DISTANCE MODELS

1994 AASHTO Stopping Sight Distance Model

According to AASHTO, the “minimum sight distance available on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary hazardous object in its path. Thus, this minimum distance is referred to as stopping sight distance (2).” Stopping sight distance is a combination of braking distance and the driver’s

brake reaction distance. Equation 4 is the 1994 AASHTO stopping sight distance model for crest vertical curves based on the design vehicle of a passenger car.

$$SSD = 0.278(V)t + \frac{V^2}{254(f \pm g)} \quad (4)$$

where,

- SSD = stopping sight distance, m;
- V = initial speed, km/h;
- t = brake reaction time, sec;
- f = coefficient of friction, dimensionless; and
- g = grade, % grade/100, m/m.

The available sight distance on crest vertical curves is established by a straight line of sight from the driver's eye in a passenger car to the top of an object in the vehicle's path. The line of sight must clear the simple parabolic vertical curve of the roadway surface. The mathematical relationship when the available sight distance is less than the horizontal length of vertical curve is displayed in Equation 5. Equation 6 shows the mathematical relationship when the available sight distance is greater than the horizontal length of vertical curve.

When S is less than L,

$$L = \frac{AS^2}{100(\sqrt{2h_1} + \sqrt{2h_2})^2} \quad (5)$$

When S is greater than L,

$$L = 2S - \frac{200(\sqrt{h_1} + \sqrt{h_2})^2}{A} \quad (6)$$

where,

- L = horizontal length of vertical curve, m;
- A = algebraic difference in grades, %;
- S = available sight distance, m;
- h₁ = height of driver's eye above roadway surface, assumed 1070 mm; and
- h₂ = height of object above roadway surface, assumed 150 mm.

In the 1994 AASHTO model, the height of the eye is assumed to be 1070 mm (3.50 ft) and the height of object in the roadway is assumed to be 150 mm (0.50 ft). The object height is based on the height of potentially hazardous debris that can be encountered on a roadway surface.

NCHRP 400 Stopping Sight Distance Model

The NCHRP 400 model has two changes in philosophy with respect to the 1994 AASHTO model. These changes apply to the braking distance and the height of the object and driver's eye. Previous research has shown that approximately 90 percent of all drivers choose deceleration rates greater than 3.40 m/sec^2 (11.15 ft/sec^2) when confronted with the need to stop for an unexpected object in the roadway (6). These decelerations allow drivers to stay within their lane while maintaining steering control during the braking maneuver and are more realistic than the braking theory in the 1994 AASHTO model (6). Therefore, a different braking distance equation was developed. In turn, the stopping sight distance was modified to account for the change in the braking distance. The brake reaction time of 2.5 seconds remains the same in the NCHRP 400 model. The stopping sight distance equation based on the NCHRP 400 model is shown in Equation 7.

$$SSD = 0.278(V)t + \frac{V^2}{254\left(\frac{a}{9.81} \pm g\right)} \quad (7)$$

where,

- SSD = stopping sight distance, m;
- V = initial speed, km/h;
- t = brake reaction time = 2.5 sec; and
- a = driver deceleration, m/s^2 .
- g = grade, % grade/100, m/m

The crest vertical curve length equations in the NCHRP 400 model are also based on a simple parabolic curve and the mathematical relationships are identical to Equations 5 and 6. The differences between the two models are the heights of the driver's eye and the object in the roadway. The driver's eye height, h_1 , is considered to be 1080 mm (3.54 ft) instead of 1070 mm (3.50 ft) above the roadway surface. This value is based on studies stating that the average vehicle heights have increased, in turn making the driver's eye height increase proportionally. The object height, h_2 , is considered to be 600mm (2.00 ft), instead of 150 mm (0.5 ft) above the roadway surface. This value is based on the very low probability of an object of 150 mm (0.5 ft) being present on the road and the drivers' ability to perceive and react to the situation. The object assumed to be in the roadway in the NCHRP 400 model is a stalled passenger car. Therefore, a value of 600 mm (2.00 ft) is assumed since it is the 85th-percentile height of passenger cars' taillights. It is rationalized that a driver could perceive, react, and stop before impacting the 600 mm (2.00 ft) object (6).

PROCEDURE

Field studies were conducted at 70 crest vertical curves on horizontal tangent sections of two-lane rural highways in Nebraska. All 70 sites had an inferred design speed less than the posted speed based on the 1994 AASHTO model. Of the 70 sites, 17 had inferred design speeds less than the posted speed, based on the NCHRP 400 model. The 70 study sites had a variety of posted speeds, shoulder widths, annual daily traffic, and vertical curve types (I or II). Type I crest vertical curves have a positive approach grade and a negative departure grade. Type II crest vertical curves have either positive approach and departure grades (approach grade steeper than departure grade) or negative approach and departure grades (departure grade steeper than approach grade). TABLE 1 shows the range of site characteristics used in the selection of the 70

study sites. The study sites were also screened to minimize the effects of extraneous factors which could possibly influence a motorist's speed choice. The following list shows the extraneous factors considered as potentially influential:

- poor pavement conditions,
- vertical grade greater than 5%,
- guardrail present on one or both sides of the roadway,
- horizontal curvature,
- bridges,
- road intersections,
- traffic control signs, and
- lane widening.

Beyond having a range of posted speeds, shoulder widths, and annual daily traffic volumes, the final 70 crest vertical curve sites decided upon had the following geometric characteristics which are summarized in TABLE 2:

- design speed,
- vertical curve length,
- K-values,
- number of vertical deficiencies per mile,
- vertical curve type,
- vertical deficiencies within a range of distances from upstream and down stream vertical deficiencies.

DATA COLLECTION

The speed data were collected between June 1999 and October 1999. The NU-METRICS NC-97 Traffic Counter/Classifier was used during the field studies. The NU-METRICS NC-97 Counter is a vehicle magnetic imaging traffic counter that uses the Earth's magnetic field in conjunction with a vehicle's magnetic mass to count and classify vehicles. The NU-METRICS NC-97 counters had the accuracy of ± 1.61 km/h (± 1 mph) in the recording of speed data. The main traffic characteristics of interest were vehicular speed and length. Other factors of interests were the time of day and pavement condition (wet/dry). These data were required in order to analyze the speed data for specific vehicles and road conditions.

Two counters were located at each crest vertical curve site. One was located where the vehicles would be expected to be operating at their desired speed. This control location was positioned at least 221 m (725 ft) from the crest of the vertical curve which is equivalent to a desirable stopping sight distance for a 105 km/h (65 mph) initial speed based on the 1994 AASHTO model. A distance of 221 m (725 ft) would translate to an equivalent stopping sight distance for a 140 km/h (87 mph) initial speed based on the NCHRP 400 model. The other counter was placed where the stopping sight distance was most limited (limit location). Several figures created by Fambro et al. (8) were used to determine these placements relative to their distance from the crest of the vertical curve. The location ranged from 46 m (150 ft) to 122 m (400 ft) from the crest on the vertical curves.

For all of the field studies, at least 275 vehicular speeds were collected to assure an adequate sample size to obtain a 95% level of confidence under free flow conditions. The sample size requirements were determined through Equation 8.

$$N = \frac{\sigma^2 K^2 (2 + U^2)}{2E^2} \quad (8)$$

where,

- N = minimum number of measured speeds;
- σ = estimated sample standard deviation (± 8.5 km/h);
- K = constant corresponding to the desired confidence level;
- E = permitted error in the average speed estimation, mph (± 1.6 km/h); and
- U = constant corresponding to the desired percentile speed.

DATA ANALYSIS

Data was collected from 70 study sites, of which 62 study sites were used in the development of the relationships and the additional 8 study sites were used during the verification process. One study site was chosen from each of the categories shown in TABLE 1. The verification site chosen to represent the 89 km/h (55 mph) category III was not considered since it was a low volume highway leading into and ending in a recreational park. The study sites used in the development of the model were tested for normality using the Chi-Squared Goodness-of-Fit Test. Most of the field sites had speed distributions that represented a normal curve at the 95% level of confidence.

The speed data were sorted by time of day, vehicle classification, and pavement condition (wet/dry). Vehicle headways of less than 5 seconds were removed leaving only free-flow speeds in the data set. Past research by Taylor and McGee (9) and Lamm et. al. (10) had found insignificant differences between day versus night speeds and wet versus dry pavement conditions, respectively. The final speed data used in the analysis procedure were passenger car speeds during daylight hours in dry conditions. These data sets were chosen since the current equation for the design of vertical curves in the United States is based on passenger cars and

there were also minimal night speeds and speeds recorded during wet conditions in the data collected.

The parameters considered as possible factors in the prediction of the mean speed, 85th-percentile speed, 95th-percentile speed were:

- approach grade,
- algebraic difference between the approach and departure grades,
- length of vertical curvature,
- K-value,
- vertical curve inferred design speed (1994 AASHTO and NCHRP 400),
- ADT,
- heavy vehicle percentage,
- posted speed,
- roadway width,
- shoulder width, and
- shoulder type.

The best-model selection procedure was used to identify the influential factors along with the backward selection process to identify statistical significance. An adjusted coefficient of multiple determination (R_a^2), Mallow's C_p , and mean squared error (MSE) was then identified for the selected models. A 95% level of confidence was used during the data analysis procedures. Influential sites were identified and removed to improve the reliability of the models. These were identified by their studentized residuals, leverage, and DFITS statistic. A total of 13 sites out of 62 sites were removed, leaving 49 sites for the analysis procedures. The same 49 sites were used for all analysis procedures.

RESULTS

Predicted Speeds at the Point of Limited Sight Distance

Mean, 85th-Percentile, and 95th-Percentile Speeds at the Limit Location

The regressed linear models for mean, 85th-percentile, and the 95th-percentile speeds had the posted speed, the approach grade, and the ADT as the factors that influence speed at the location on a crest vertical curve with limited stopping sight distance. These relationships are shown in Equations 9, 10, and 11. TABLE 3 shows the statistical parameters based on a 95% level of confidence.

$$V_{\text{mean, limit}} = 68.5 + 0.382V_p - 0.703G_1 - 0.00174T_{\text{ADT}} \quad (9)$$

where,

$V_{\text{mean, limit}}$	=	average speed of passenger car at the point of limited stopping sight distance, km/h;
V_p	=	posted speed, km/h;
G_1	=	approach tangent grade, %; and
T_{ADT}	=	ADT (valid for $\text{ADT} \leq 5000$)

$$V_{85, \text{limit}} = 73.9 + 0.400V_p - 0.124G_1 - 0.00143T_{\text{ADT}} \quad (10)$$

where,

$V_{85, \text{limit}}$	=	85 th -percentile speed of passenger cars at the point of limited stopping sight distance, km/h.
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$$V_{95, \text{limit}} = 85.5 + 0.346V_p - 0.0749G_1 - 0.00181T_{\text{ADT}} \quad (11)$$

where,

$V_{95, \text{limit}}$	=	95 th -percentile speed of passenger cars at the point of limited stopping sight distance, km/h.
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The observed and predicted mean, 85th-percentile, and 95th-percentile speeds are listed in TABLE 4. The observed and predicted speed values were analyzed with a paired t-test within each individual speed parameter. The resulting p-value for each speed parameter was greater than 0.05, therefore there was not a statistically significant difference between the observed and predicted mean, 85th-, and 95th-percentile speeds at the 95% level of confidence.

Predicted Speeds at the Control Location

Mean, 85th-Percentile, and 95th-Percentile Speeds at the Control Location

The speed parameters obtained at the control location were not considered during the development of the model at the location of limited stopping sight distance. Therefore, a paired t-test was performed on the mean, 85th-percentile, and 95th-percentile speeds between the limit and control locations to determine if the speeds differ. All of the p-values were found to be greater than 0.05. Therefore, the mean, 85th-percentile, and 95th-percentile paired speeds at the control and limit locations were not significantly different at the 95% level of confidence.

Since the findings of the paired t-test resulted in the conclusion that there was no difference in speeds between the control and limit locations, multiple linear regression was performed on the control location data to determine the factors that influence the operating speed parameters at the control location. The independent variables considered as possibly being statistically significant in the prediction of the mean, 85th-percentile, and 95th-percentile speeds at the control location were:

- ADT,
- heavy vehicle percentage,
- posted speed,
- roadway width,

- shoulder width, and
- shoulder type.

The operating speed relationships found for the control location are shown in Equations 12, 13, and 14. The statistical summaries of the regression equations are shown in TABLE 5.

$$V_{\text{mean, control}} = 55.4 + 0.496V_p - 0.00147T_{\text{ADT}} \quad (12)$$

where,

$$\begin{aligned} V_{\text{mean, control}} &= \text{mean speed of passenger cars at control location, km/h;} \\ V_p &= \text{posted speed, km/h; and} \\ T_{\text{ADT}} &= \text{ADT.} \end{aligned}$$

$$V_{85, \text{control}} = 83.1 + 0.307V_p - 0.00141T_{\text{ADT}} \quad (13)$$

where,

$$V_{85, \text{control}} = 85^{\text{th}}\text{-percentile speed of passenger cars at control location, km/h.}$$

$$V_{95, \text{control}} = 90.5 + 0.285V_p - 0.00119T_{\text{ADT}} \quad (14)$$

where,

$$V_{95, \text{control}} = 95^{\text{th}}\text{-percentile speed of passenger cars at control location, km/h.}$$

TABLE 6 shows the results of the paired t-test between the observed and predicted speeds for the verification sites at the control location for the mean, 85th-, and 95th-percentile speeds. All of the p-values were greater than 0.05, which indicated that there were no significant differences between the predicted and observed mean, 85th-percentile, and 95th-percentile speeds for verification at the 95% level of confidence.

The independent variables used for the prediction of speeds at the limit locations (available stopping sight distance less than 1994 AASHTO minimum criteria) did not include any speed parameters of the control location, but the models at both the control and limit

locations include posted speed and ADT as significant variables. The speeds at locations with less than minimum stopping sight distance were also influenced by the approach grade.

Analysis of Sites Using Previously Established Equations for Estimated Operating Speeds on Crest Vertical Curves

Equations developed by Fambro et al. (Equation 1) and Fitzpatrick et al. (Equations 2 and 3) were used to predict 85th-percentile speeds at the Nebraska vertical curve equation validation sites. The accuracy of these equations was compared to the prediction accuracy of the equations developed in this research on the seven validation sites.

For sites with vertical crest curvature with non-limited sight distance ($K > 43 \text{ m/\%}$) ($K > 141 \text{ ft/\%}$) on a horizontal tangent, an assumed desired speed of 100 km/h (62.1 mph) was used. Fitzpatrick et al. suggest this speed to be a good estimate of the desired speed along a two-lane rural roadway when seeking a representative rounded speed that was determined from their research (6).

Only two verification sites fit the shoulder width criteria for Fambro's equation. The mean squared error was calculated for the difference between the observed 85th-percentile value and the following equations at these two sites:

- Fambro's equation,
- 85th-percentile speed from the regressed equation developed in this research, and
- Fitzpatrick's equation.

Results of the accuracy comparison are shown in TABLE 7. The regressed equation had the best fit of all equations compared.

Only two of the seven sites had $K \leq 43 \text{ m/\%}$ ($K \leq 141 \text{ ft/\%}$). The appropriate Fitzpatrick equation was used at these two sites. The remaining sites were non-limited site distance

locations as defined by Fitzpatrick so a desired speed was considered to be 100 km/h (62.1 mph). The mean squared error was calculated for the difference between the observed 85th-percentile value and the following equations at all 7 sites:

- 85th-percentile speed from the regressed equation developed in this research, and
- Fitzpatrick's equation.

Results of the accuracy comparison of the Fitzpatrick et al. equations versus the equation developed by this research are shown in TABLE 8. The MSE comparison shows that the regressed equation provides the best fit of all those used to predict the 85th-percentile speed for the verification sites.

Design Speed

The design speed of a vertical curve is the maximum speed that allows a safe stop according to equations from the 1994 AASHTO Green Book or NCHRP 400. The 1994 AASHTO Green Book implies that the desirable design speed should approximately correspond to the 95th-percentile speed of the cumulative distribution of speeds at a given location (2). Ideally, the design speed should equal the observed 95th-percentile vehicular speeds on crest vertical curves according to the 1994 AASHTO Green Book philosophy.

The observed 95th-percentile speeds at the limit locations versus the design speed are shown in FIGURES 1 and 2. The design speed on the x-axis in FIGURE 1 is based on the 1994 AASHTO model and the design speed on the x-axis in FIGURE 2 is based on NCHRP 400 model. FIGURE 1 indicates that the 1994 AASHTO model provides design speeds that are much lower than the 95th-percentile speeds observed on crest vertical curves. An ideal scenario would be to have all the data points on or below the solid line indicating that 95 percent of the

drivers travel at or below the design speed. However, most of the 95th-percentile speeds were found to be 10 to 30 mph above the inferred design speed.

FIGURE 2 shows that the NCHRP 400 design speed model predicts design speeds closer to the observed 95th-percentile operating speeds than the 1994 AASHTO model. The majority of the 95th-percentile speeds in FIGURE 2 are only 0 to 24 km/h (15 mph) above the design speed.

CONCLUSIONS

In this study, 70 crest vertical curve sites on horizontal tangents were chosen specifically because the inferred design speed was less than the posted speed according to the 1994 AASHTO model. However, according to the NCHRP 400 model, only 17 of the 70 had an inferred design speed less than the posted speed limit. Regression equations were developed to predict the mean, 85th-, and 95th-percentile speed of passenger cars in dry, daylight conditions at limited stopping sight distance locations approaching crest vertical curves. Prediction equations were also developed for the same speed parameters at locations with desirable stopping sight distance where it is assumed that drivers are traveling at their desired speed. The developed equations for operating speeds at the location of limited stopping sight distance and control locations are shown in Equations 9, 10, and 11 and Equations 12, 13, and 14, respectively.

The approach grade affected vehicle speeds at the limit location. As the approach grade increased, the mean, 85th-, and 95th-percentile speeds decreased. At both the limit and control locations, the posted speed of the roadway facility had the most influence on mean, 85th-, and 95th-percentile speeds. As the ADT increased, the mean, 85th-, and 95th-percentile speeds decreased at the limit and control locations indicating that motorists may view increases in volume as motivation to slow down. Based on the significant parameters in the developed equations, the crest vertical curve characteristics do not influence vehicle speeds when

approaching a point along the curve where the stopping sight distance is less than the minimum 1994 AASHTO criteria.

The NCHRP 400 model was found to be more realistic than the 1994 AASHTO model, because it yielded more design speeds greater than the 95th-percentile speeds observed on highways. Also, the model resulted in less variance between the design speeds and 95th-percentile speeds.

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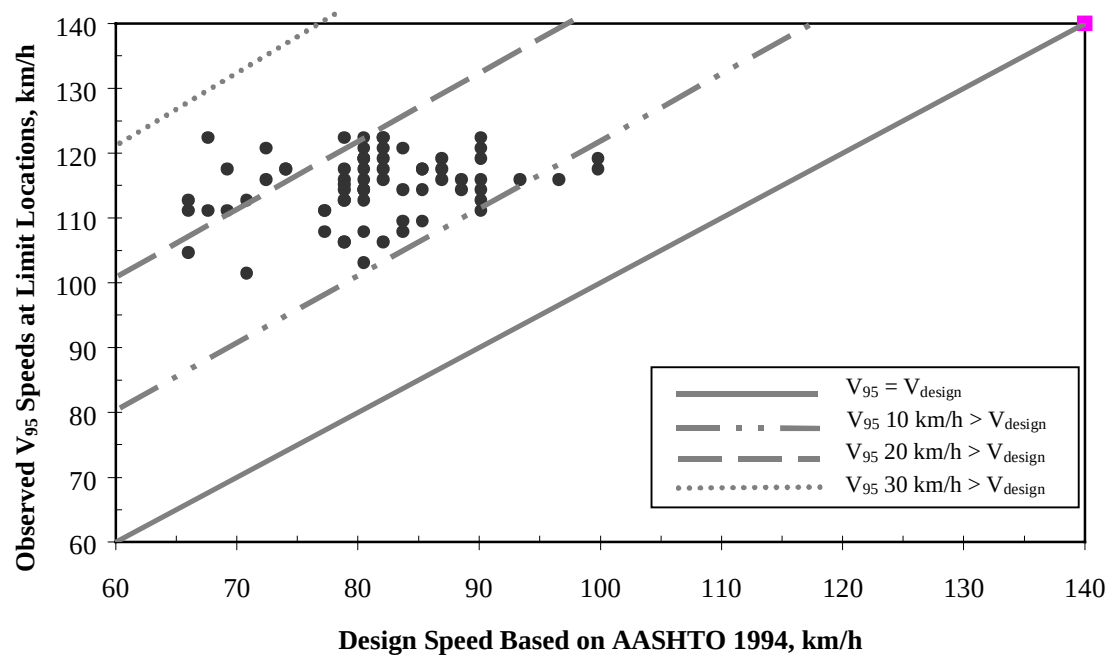


FIGURE 1 Observed V_{95} versus 1994 AASHTO Design Speed Model.

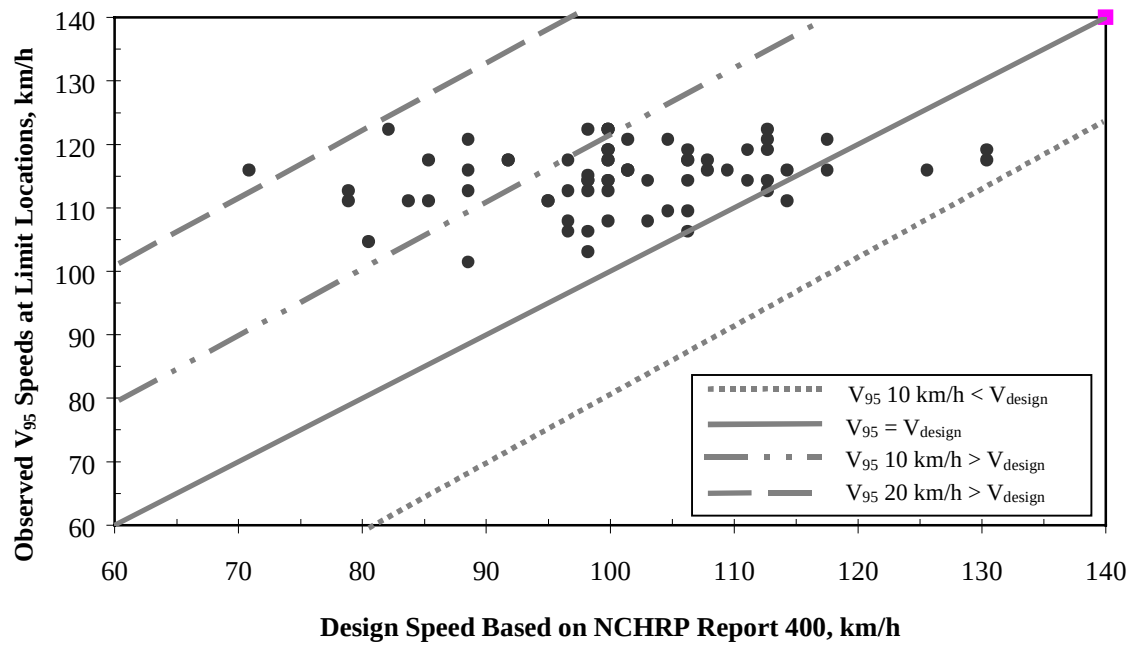


FIGURE 2 Observed V_{95} versus NCHRP 400 Design Speed Model.

TABLE 1 Final Categories Used for Site Selection

Posted Speed (km/h)	Category	Crest Vertical Curve Type		Shoulder Width (m)	Annual Daily Traffic (ADT)
		I	II		
89	I	7	3	1.2	1700 – 2200
89	II	5	5	0.6 – 1.8	910 – 1350
89	III	2	4	0.3 – 1.7	110 – 225
97	I	7	2	3.0	3424 – 4086
97	II	3	5	1.5 – 2.4	635 – 1380
97	III	4	4	0.3 – 1.8	80 – 415
105	I	4	5	2.7 – 3.0	1700 – 3280
105	II	5	5	1.5 – 2.4	625 – 1596
105	III	There are no U.S. or Nebraska Highways posted at 105 km/h with volumes less than 400 ADT			

TABLE 2 Summary of Characteristics of Vertical Curve Sites

Control	Criteria
Area Type	Rural
Functional Classification	Collector or minor arterial
Design Speed (1994 AASHTO)	60 km/h – 100 km/h
Design Speed (NCHRP 400)	71km/h – 132 km/h
Posted Speed Limit	89 km/h – 105 km/h
Approach Grade	-3.30% to +4.98%
Departure Grade	-7.00% to +1.65%
Algebraic Difference	1.69 – 10.57
K	24.7 m – 108.2 m
Vertical Curve Length	91.4 m – 396.2 m
Traffic Volumes	80 – 4086 vehicles per day
Roadway Width	6.1 m – 7.3 m
Shoulder Widths	0.3 m – 3 m
Shoulder Surface Widths	0 m – 2.4 m

TABLE 3 Statistical Summary of the Variables at the Limit Location

Independent Variable	Coefficient	T-Statistic	p-value	R _a ²	C _p	MSE
Mean Speed (Equation 9)						
Intercept	42.2	10.0	0.0000	0.57	1.78	3.51
G ₁	-0.451	-2.75	0.0087			
T _{ADT}	-0.00109	-4.77	0.0000			
V _P	0.388	5.61	0.0000			
85 th -Percentile Speed (Equation 10)						
Intercept	53.9	11.8	0.0000	0.55	3.34	4.11
G ₁	-0.405	-2.28	0.0277			
T _{ADT}	-0.00150	-6.06	0.0000			
V _P	0.301	4.01	0.0002			
95 th -Percentile Speed (Equation 11)						
Intercept	62.0	15.0	0.0000	0.57	1.41	3.39
G ₁	-0.427	-2.65	0.0111			
T _{ADT}	-0.00148	-6.63	0.0000			
V _P	0.224	3.30	0.0019			

TABLE 4 Observed versus Predicted Mean, 85th-, and 95th-Percentile Speed at Limit Location

Highway	Reference Post	Significant Independent Variables			Observed (km/h)	Predicted (km/h)
		G ₁ (%)	T _{ADT}	V _p (km/h)		
Mean Speed (Equation 9)						
NE-9	14.44 SB	-0.23	1380	97	100	103
NE-18	13.58 WB	4.75	80	97	95	101
US-20	88.71 EB	1.62	2184	105	103	103
US-20	145.04 EB	2.86	625	105	101	105
NE-33	15.74 EB	-0.28	3424	97	95	100
NE-35	19.48 EB	0.00	2200	89	100	98
NE-121	71.93 NB	-0.04	1350	89	103	100
85 th -Percentile Speed (Equation 10)						
NE-9	14.44 SB	-0.23	1380	97	108	113
NE-18	13.58 WB	4.75	80	97	108	109
US-20	88.71 EB	1.62	2184	105	113	113
US-20	145.04 EB	2.86	625	105	109	114
NE-33	15.74 EB	-0.28	3424	97	103	109
NE-35	19.48 EB	0.00	2200	89	108	108
NE-121	71.93 NB	-0.04	1350	89	114	109
95 ^h -Percentile Speed (Equation 11)						
NE-9	14.44 SB	-0.23	1380	97	114	119
NE-18	13.58 WB	4.75	80	97	116	118
US-20	88.71 EB	1.62	2184	105	118	118
US-20	145.04 EB	2.86	625	105	116	119
NE-33	15.74 EB	-0.28	3424	97	108	113
NE-35	19.48 EB	0.00	2200	89	113	114
NE-121	71.93 NB	-0.04	1350	89	121	116

TABLE 5 Statistical Summary of the Significant Variables at the Control Location

Independent Variable	Coefficient	T-Statistic	p-value	R_a²	C_p	MSE
Mean Speed (Equation 12)						
Intercept	34.5	6.74	0.0000			
T _{ADT}	-0.000928	-3.25	0.0022	0.44	1.31	5.62
V _P	0.498	5.80	0.0000			
85th-Percentile Speed (Equation 13)						
Intercept	44.6	8.21	0.0000			
T _{ADT}	-0.00129	-4.26	0.0001	0.42	1.61	6.33
V _P	0.437	4.80	0.0000			
95th-Percentile Speed (Equation 14)						
Intercept	51.2	10.1	0.0000			
T _{ADT}	-0.00124	-4.38	0.0001	0.41	1.69	5.56
V _P	0.384	4.49	0.0000			

TABLE 6 Observed versus Predicted Mean, 85th-, and 95th-Percentile Speeds at the Control Location

Highway	Reference Post	Significant Independent Variables		Observed (km/h)	Predicted (km/h)
		T _{ADT}	V _p (km/h)		
Mean Speed (Equation 12)					
NE-9	14.44 SB	1380	97	105	101
NE-18	13.58 WB	80	97	97	103
US-20	88.71 EB	2184	105	97	105
US-20	145.04 EB	625	105	109	106
NE-33	15.74 EB	3424	97	100	98
NE-35	19.48 EB	2200	89	93	97
NE-121	71.93 NB	1350	89	100	98
85 th -Percentile Speed (Equation 13)					
NE-9	14.44 SB	1380	97	113	114
NE-18	13.58 WB	80	97	109	109
US-20	88.71 EB	2184	105	105	116
US-20	145.04 EB	625	105	118	111
NE-33	15.74 EB	3424	97	111	109
NE-35	19.48 EB	2200	89	101	108
NE-121	71.93 NB	1350	89	109	111
95 ^h -Percentile Speed (Equation 14)					
NE-9	14.44 SB	1380	97	118	118
NE-18	13.58 WB	80	97	114	119
US-20	88.71 EB	2184	105	109	118
US-20	145.04 EB	625	105	122	121
NE-33	15.74 EB	3424	97	114	113
NE-35	19.48 EB	2200	89	106	113
NE-121	71.93 NB	1350	89	116	114

TABLE 7 Comparison of Developed Equations with Equations from Previous Research at Locations with Shoulders Less Than 1.8 m (6 ft) (Verification Sites Only)

Highway	Ref. Post	Comparisons of 85 th -Percentile Speeds (km/h)			
		Observed 85 th -Percentile Speed	85 th -Percentile Speed Regressed Equation 10	Fambro et al. V ₈₅ Equation 1	Fitzpatrick et al. V ₈₅ Equation 3
NE-18	13.58 WB	108	112	115	100
NE-35	19.48 EB	108	106	104	100
		MSE =	9.65	32.0	62.3

TABLE 8 Comparison of Developed Equations with Equations from Fitzpatrick et al. (25)

Highway	Ref. Post	Comparisons of 85 th -Percentile Speeds (km/h)		
		Observed 85 th -Percentile Speed	85 th -Percentile Speed Regressed Equation 10	Fitzpatrick et al. V ₈₅ Equation 2
NE-9	14.44 SB	108	111	100
NE-18	13.58 WB	108	112	100
US-20	88.71 EB	113	112	100
US-20	145.04 EB	109	115	100
NE-33	15.74 EB	103	108	100
NE-35	19.48 EB	108	106	100
NE-121	71.93 NB	114	108	101
		MSE =	17.5	88.6