Chapter 1
INTRODUCTION

Background
Ideally, the angle of intersection between two roadways should be as near to 90 degrees as practical. This angle optimizes safety in the following respects:

- The conflict area between vehicles is the smallest,
- Viewing range from left to right is least restricted,
- Paved intersection surfacing for trucks is least, and
- Exposure for motorists and pedestrians to opposing traffic is least.

Due to varying existing horizontal alignments of roadways, some intersections are skewed (i.e., roadways intersect at angles other than 90 degrees). At a rural two-way stop-controlled intersection, a common design solution to eliminate a skewed intersection is to introduce a horizontal curve in the stop-controlled roadway approach in order to create an optimal 90-degree angle. Some examples of different realignment patterns are shown in Figure 1.1 (*1*). In each case, a short tangent section is provided between the horizontal curve and the intersection. This is provided to allow for superelevation transition near the intersection.

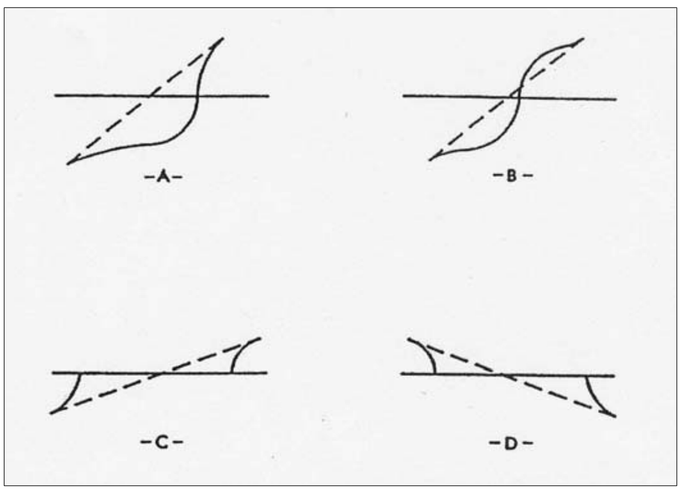

Figure 1.1. Methods for Aligning Skewed Intersections (*1*)

In general, the radius chosen for a horizontal curve is dependent upon the design speed of the roadway, the maximum allowable superelevation and applicable side friction factor developed between the contact patch of the tires and the pavement surface. However, in the situation where the curve is located very near a stop, a vehicle may be traveling at a constantly changing speed due to the necessity to decelerate to a stop. This fact makes the choice of horizontal alignment a challenging one for roadway designers. With limited guidance provided to designers on the subject, the problem of design consistency is called into question. If designers are not given guidance as to how curves that transition speeds from high values to a stop condition should be designed, they will design the curve as they see fit, according to their own state's standards and practices, if any are available. This variance in design may lead to a violation of driver expectancy.

Driver expectancy refers to the evaluation and memory of successful responses to situations based on past driving experiences. A priori expectancy is based on years of driving experience. It is important that designers create geometric features that conform to driver expectancy. This will result in fewer driver errors and an increase in safety. If design procedures were appropriate and available, the design of horizontal curves approaching a stop would become more uniform. A uniform or consistent design is desirable because it conforms to driver expectations. Research has found that if a road is consistent in design, then it should not inhibit the ability of motorists to control their vehicle safely (*2*). Also, consistent roadway design ensures that "most drivers would be able to operate safely at their desired speed along the entire alignment (*3*)."

Objective

The objective of this research is to develop a model that describes the operating speed profiles of vehicles traversing horizontal curves on approaches to stop-controlled intersections on rural two-lane two-way highways. This model would allow the prediction of the operating speed of a vehicle at a given distance from the stop line. Once a speed prediction model is determined, a procedure for the design of horizontal curves on rural highways that must accommodate vehicles transitioning from high speeds to a stop-controlled intersection can be developed.

Literature Review

The guidelines currently used by the Nebraska Department of Roads (NDOR) to design curves on stop-controlled approaches at skewed intersections include the following statement: "The superelevation of stop-controlled approaches on curved alignments should be flattened to allow vehicles to retain control during slowing and stopping. The superelevation should accommodate a design speed of 30 km/h (20 mph) less than required, but should not be less than 50 km/h (30 mph). This will accommodate a reasonable operating speed while minimizing the potential for adverse operations under wet driving conditions. A short tangent section should be provided on the approach to allow for superelevation runoff" (*1*).

The following is a summary of the American Association of State and Highway Transportation Officials (AASHTO) design policy recommendations on the subject. The AASHTO 2001 guide book, "A Policy on Geometric Design of Highways and Streets", hereafter referred to as the Green Book states: "The speeds for which these intersection curves should be designed depend on vehicle speeds on the approach highways, the type of intersection, and the volumes of through and turning traffic. Generally, a desirable turning speed for design is the average running speed of traffic on the highway approaching the turn."(*4*) In previous studies regarding the speed-curvature relationship, the 95th-percentile speed of traffic was used as the design speed. Relationships between design speed (95th-percentile) and side friction factor were established for rural and high-speed urban curve design for at-grade intersections. For design of intersection curves it is desirable to establish a single minimum radius for each design speed by assuming a likely minimum rate of superelevation that can nearly always be obtained for certain radii. If more superelevation than this minimum is actually provided, drivers will either be able to drive the curves a little faster or drive them more comfortably because of less friction (*4*).

A common feature in the NDOR and AASHTO design policies is the use of superelevation to control the design speed of the curved roadway, the goal being to provide a slope that allows passenger cars and trucks to travel comfortably without adverse lateral acceleration or skidding and roll over.

The study of the relationship between horizontal curvature and speed or accidents has been extensive. However, most of this research has not been focused on curves near stop-controlled intersections. The literature did point out some issues that were investigated during the course of this study. For example, the differences between the performance of passenger cars and heavy vehicles are sometimes overlooked in the design of horizontal curves. Harwood found that for design speeds of 10 to 20 mph, a truck could skid or roll over by exceeding the design speed of a minimum-radius curve by 4 mph or less (*5*). Another important item when investigating unique sites is to determine if they have higher accident rates than comparable sites. In Fink, et al. it was found that degree of curvature is a good predictor of accident rates (*6*). Finally, Andjus concluded that one of the main concerns for a road designer should be how a driver will respond to elements designed according to a specific standard speed through the speed adjustments of the vehicle while using the road in question (*7*). These three sources are important because heavy vehicles are a significant proportion of the vehicle population in Nebraska. In addition, all roadway characteristics should be studied to determine their influence on driver performance.

Chapter 2
STUDY SITES

Candidate Sites

The initial goal of the research was to collect data at 50 study sites. The site selection process began by looking for highway intersections on the state highway map that appeared to either be skewed or have a curve near the intersection. Once 75 candidate sites had been identified, the video log at the NDOR was used to further investigate the horizontal alignment and roadside features at each site. Sites were eliminated from consideration if they didn't contain a horizontal curve near the intersection, or had railroad crossings or other non-typical features, such as free right-turn lanes in the opposing direction near the intersection that might influence vehicle speeds approaching a stop. Aerial photographs provided by the United States Geological Survey (USGS) were used to view the curve with respect to its surroundings (*8*).

The three most common types of horizontal curve alignments are single or simple, reverse, and compound. The most basic curve, the simple curve, has only one curve which is preceded and followed by a tangent length of roadway. A reverse curve is a roadway section that consists of two curves on opposite sides of a common tangent with a relatively short tangent between them (*1*). A compound curve consists of two consecutive curves which join on the same side of a common tangent with no tangent length between them (*1*). In the case of the reverse and simple curves, the tangent length is necessary to allow for the development and runoff of superelevation.

Once qualifying sites were selected, supplemental information was gathered using the 2000 State Highway Inventory Report prepared by the NDOR (*9*). The inventory data were used to identify possible factors that would influence vehicle speeds. These data included average daily traffic (ADT), percent trucks, surface type, surface condition, shoulder width, and accidents. Surface condition was characterized by three categories. One category was the Nebraska Serviceability Index (NSI), which ranks roadway conditions from 0 to 100 with 0 being the worst and 100 being the best. The second category was the Performance Serviceability Index (PSI), which is an AASHTO index that ranks roadways from 0 to 5 (0 being the worst and 5 being the best) based on the functional ability of the pavement to serve the traveling public. The third category was rutting, which is the average rut depth for bituminous pavement measured in millimeters. Table 2.1 summarizes the surface condition ratings for the state of Nebraska based on these three factors. Shoulder width included total shoulder width and paved width.

Table 2.1. Roadway Condition Standards for Nebraska

Description	NSI	PSI	Rutting
Very Good	90 and Over	4.1 to 5.0	
Good	70 thru 89	3.1 to 4.0	< 0.24 in.
Fair	50 thru 69	2.1 to 3.0	0.24 in. thru 0.51 in.
Poor	30 thru 49	1.1 to 2.0	> 0.51 in.
Very Poor	0 thru 29	0.0 to 1.0	

Source: (*9*)

Official accident reports were requested for sites with a high number of accidents which were reviewed to discover a possible link between horizontal alignment and safety. More detailed information about each site was gathered using archived as-built plan and profile sheets from the NDOR. From the plans, the location of the point of intersection (PI), point of tangency (PT), and point of curvature (PC) were found, along with the deflection angle (Δ), degree of curve (D), tangent length, radius, length of curve, and where available, maximum superelevation rates. Actual superelevation along the horizontal curves at study site locations was also field measured. Actual maximum superelevation values were used to estimate the inferred curve design speed as follows:

$$V_c = (14.90R(e + f\,))^{0.5} \tag{2.1}$$

where,

V_c = inferred curve design speed (mph),
e = actual maximum superelevation rate (ft/ft),
f = side friction factor (from Table 2.2), and
R = radius of curvature of the traveled path (ft).

Table 2.2. Maximum Design Side Friction Factors

Design Speed (mph)	Maximum Side Friction Factor f_{max}
20	0.22
30	0.19
40	0.16
50	0.13
55	0.12
60	0.11
65	0.10
70	0.09
75	0.08

Source: (*10*)

To study design and operating characteristics relating to horizontal curves on roadway alignments, design speed and posted speed need to be clearly defined. Current AASHTO design policy defines design speed as “the selected speed used to determine the various geometric design features of the roadway” (*4*). Bonneson defines “curve design speed” as the expected 95^{th}-percentile speed of freely flowing passenger cars on a horizontal curve (*10*). Posted speed is the legal speed limit on the roadway. It is usually set close to the 85^{th}-percentile speed and according to the NDOR design guidelines is generally from 5 to 10 mph slower than the design speed (*1*).

Superelevation is simply defined as the cross slope of the roadway or traveled lane. It is usually described by the change in elevation between the centerline and the edge of lane, in feet, divided by the width of the lane, in feet. A desirable tangent section of roadway in Nebraska has a superelevation of plus or minus 0.02 ft/ft or 2 percent. This minimum superelevation allows for adequate drainage of the roadway surface.

Maximum superelevation, usually 6 to 8 percent, is provided on curved roadways in order to offset lateral acceleration caused by the curve.

Selected Sites

The research presented in this report was based on the data collected at 15 sites. The location of the study sites is shown in Figure 2.1. The study site characteristics found during preliminary investigations are listed in Table 2.3. The information gathered from the NDOR inventory report is presented in Table 2.4.

Three of the selected sites were on tangent approaches to stop-controlled intersections. These three sites 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, a reverse curve, or a compound curve in the roadway alignment as it approached a stop. The horizontal curve data for these 12 sites are shown in Table 2.5. All types of sections were selected to determine if and how horizontal curves influenced the vehicle speed profiles.

Initially, it was anticipated that the study sites could be grouped by their common characteristics, a certain number of sites from each group would be studied, and a prediction equation based upon different variables could then be created using some selection techniques. However, each site fit into a number of different categories and very few sites had all of the same characteristics. Due to the wide variety of site influences, the study approach was to collect the speed data for each site separately and then to test the speed profile regression lines developed to best explain the speed/distance relationships for statistical differences.

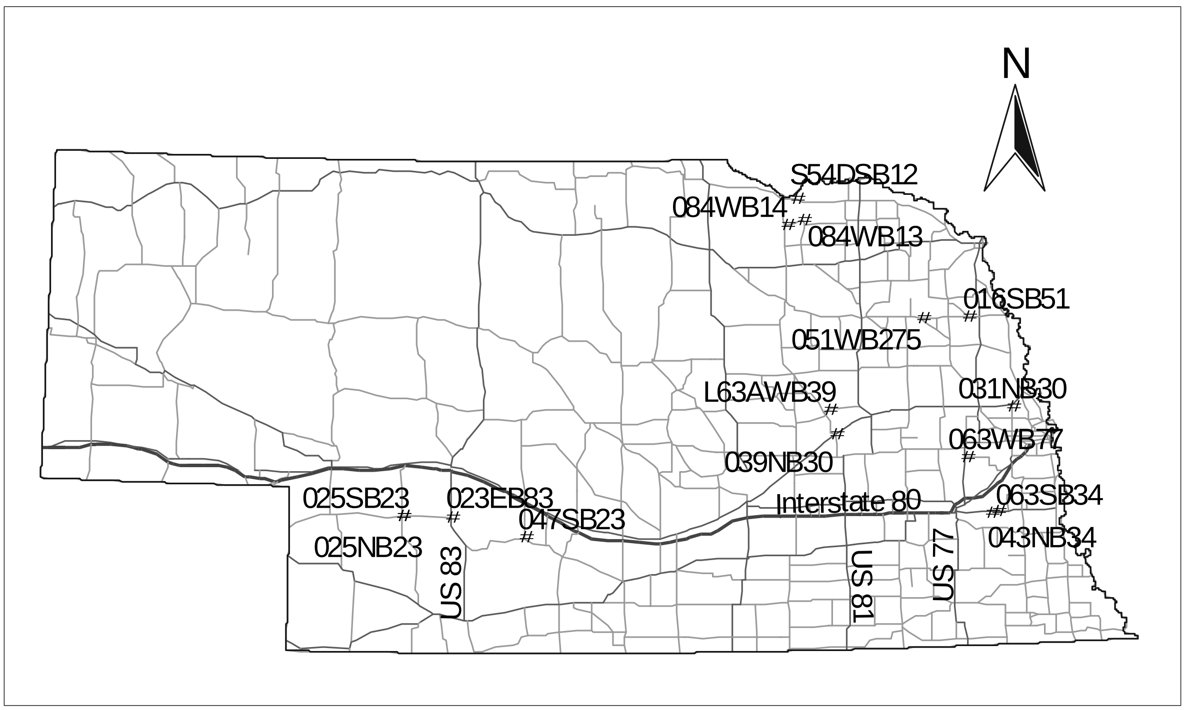

Figure 2.1. Study Site Locations

Table 2.3. Study Sites

Site	Location	County	Ref. Post	Posted Speed (mph)	Inferred Design Speed (mph)	Curve Type	Max e (ft/ft)	Rumble Bars	Median at Intersection	*Roadway Conditions (NSI & PSI)	*Rutting
016SB51	S. of Bancroft	Cuming/Burt	0+00	60	22	Reverse	0.02	None	None	Very Good	Good
023EB83	N. Wellfleet	Lincoln	70+45	60	69	Reverse	0.063	Yes	Painted	Good	Good
025NB23	E. of Wallace	Lincoln	63+24	60	45	Simple	0.042	None	None	Fair	Fair
025SB23	E. of Wallace	Lincoln	63+24	55	55	Simple	0.055	None	None	Fair	Good
031NB30	S. of Kennard	Washington	36+36	60	54	Simple	0.083	None	Raised	Good	Good
039NB30	Silver Creek	Merrick	9+78	60	51	Simple	0.052	Yes	None	Good	Good
047SB23	E. of Farnam	Dawson	47+08	60	72	Simple	0.068	Yes	None	Good	Fair
051WB275	N. of Wisner	Cuming	0+00	60	68	Simple	0.068	Yes	Raised	Very Good	Good
084WB13	S. of Center	Knox	10+34	55	51	Simple	0.057	Yes	Raised	Good	Good
084WB14	Verdigre	Knox	0+00	55	85	Compound	0.045	Yes	Raised	Good	Good
L63AWB39	Genoa Bypass	Nance	3+19	55	22	Simple	0.02	None	Raised	Fair	Good
S54DSB12	Santee Spur	Knox	0+00	55	56	Compound	0.063	Yes	None	Good	Good
043NB34	Eagle	Cass	30+27	60	NA	None	NA	None	Raised	Good	Good
063SB34	S. of Alvo	Cass	0+00	55	NA	None	NA	None	None	Good	Good
063WB77	N. of Ceresco	Saunders	38+94	60	NA	None	NA	Yes	None	Good	Fair

* See Table 2.1

Table 2.4. Inventory Report Data

Site	Present ADT	% Trucks	Surface Type	Traveled Way Width (ft)	Total Shoulder (ft)	Paved Shoulder (ft)	NSI	PSI	Rutting (in)	2000 Accident Data			5 year Accident Data		
										Fatal	Injury	Property Damage	Fatal	Injury	Property Damage
016SB51	827	11	ACC	24	5.0	0.0	94	4.2	0.12	0	0	3	2	1	3
023EB83	410	28	ACC	24	5.0	1.0	71	3.8	0.20	0	0	0	0	0	0
025NB23	465	15	ACC	24	4.0	0.0	57	4.1	0.24	0	0	4	0	0	1
025SB23	560	13	ACC	25	4.5	0.0	52	2.0	0.16	0	1	0	0	0	1
031NB30	1800	8	PCC	24	10.0	8.0	70	2.6	NA	0	0	0	0	0	1
039NB30	885	12	ACC	24	8.0	0.0	79	3.0	.012	0	0	0	0	0	0
047SB23	500	13	ACC	24	7.0	1.0	76	3.5	0.43	0	0	0	2	0	0
051WB275	1000	23	ACC	24	6.0	0.0	92	4.1	.020	0	0	1	0	1	2
084WB13	560	15	ACC	24	8.0	0.0	85	3.9	0.16	0	0	0	0	0	0
084WB14	383	11	ACC	24	6.0	0.0	88	4.0	0.16	0	1	3	0	0	1
L63AWB39	1035	13	ACC	24	2.0	0.0	60	2.4	.020	0	0	2	0	1	1
S54DSB12	570	3	ACC	24	5.0	0.0	75	3.4	0.12	0	3	1	2	1	2
043NB34	1725	8	ACC	24	4.0	0.0	96	4.2	0.12	0	2	0	0	0	1
063SB34	1045	6	ACC	24	5.0	0.0	94	4.1	0.08	0	0	2	0	0	1
063WB77	952	13	ACC	24	6.0	0.0	91	4.0	0.24	0	1	1	4	1	1

Table 2.5. Horizontal Curve Data

Site	PI STA (ft)	PT STA (ft)	PC STA (ft)	Δ (°)	Direction	D (°)	Tangent (ft)	Radius (ft)	Length (ft)
016SB51	1008+08.0	1008+77.1	1007+18.2	66.5	RT.	41.8	89.8	137.0	159.0
	217+86.2	210+18.5	225+18.5	31.0	LT.	2.0	767.7	2864.8	1500.0
023EB83	1541+36.1	1545+78.6	1536+79.3	24.7	LT.	2.8	456.8	2083.5	899.4
	1518+71.4	1523+18.5	1514+09.5	25.0	RT.	2.8	461.9	2083.5	909.0
025NB23	123+44.0	125+76.8	120+91.8	38.8	RT.	8.0	252.2	716.2	485.0
025SB23	1037+75.6	1040+42.1	1034+98.7	27.2	RT.	5.0	276.9	1145.9	543.4
031NB30	903+10.7	908+42.7	895+09.4	80.0	LT.	6.0	801.3	954.9	1333.3
	Spiral Curve	CS STA - ST STA	TS STA - SC STA	Δ - Δ_c					L_c - L_s
039NB30	533+91.2	537+17.3 - 540+67.3	525+81.9 - 529+31.9	62.5 - 43.2	RT.	5.5	809.3	1041.74	785.5 - 350.0
047SB23	1119+28.1	1125+27.9	1112+97.9	36.8	RT.	2.5	630.2	2291.8	1230.0
051WB275	73+45.3	64+50.3	81+24.2	50.2	LT.	3.0	895.0	1909.9	1673.9
084WB13	4+14.3	0+95.7	7+10.7	36.9	RT.	6.0	318.6	954.9	615.0
084WB14	25+92.6	21+36.1	30+42.7	13.6	RT.	1.5	456.4	3819.7	906.6
	39+12.3	33+81.3	44+26.1	25.2	RT.	2.4	531.1	2370.9	1044.9
L63AWB39	5001+61.4	5000+70.0	5002+44.0	43.5	LT.	25.0	91.4	229.2	174.0
S54DSB12	17+79.5	13+84.8	21+44.8	38.0	RT.	5.0	394.7	1145.9	760.0
	32+97.5	29+47.1	36+27.1	34.0	RT.	5.0	350.5	1145.9	680.0

Site Reconnaissance

To get a complete idea of the vehicle characteristics on a highway with a horizontal curve approaching a stop, data from multiple field sources were collected. Site observations regarding the condition of the surface and whether rutting existed were made at each location. Also, possible influential factors, such as guardrail, vertical alignment, access points, bridges, lane widening, rumble bars, and medians, were recorded and located with respect to the curve. In addition to these, the roadside signs on the approach were referenced to the stop line and displayed on similar drawings shown in Appendix A. Since the design speed of curves is partially dictated by superelevation, roadside cross section slopes were recorded at each detector location. As shown in Figure 2.2, a self-leveling level and Philadelphia Rod were used to find the relative elevation at the centerline, edge of lane, edge of pavement and edge of turf shoulder. The lane widths and shoulder widths were measured to get an accurate cross-slope of the roadway.

Figure 2.2. Field Measurements of Roadway Cross Slopes

Finally, a digital still photo was taken at each detector location. The photo was taken from the middle of the study lane looking in the direction of travel at a standing eye level (approx. 5.5 feet). In addition, photos were taken from the stop line looking in both directions of the intersecting highway to show the sight distance available for vehicles at the intersection. The progression of photos for each study site is presented in Appendix A.

Chapter 3
SPEED STUDIES

Data Collection

Vehicle speed data were collected between June and October 2001. Data collection time periods ranged from one to two days depending upon traffic volumes on the roadway section. Studies were conducted during favorable weather and planned not to coincide with NDOR roadway maintenance activities. The number of detectors used at each site ranged from 6 to 14 depending upon the lengths of the curve and lengths of tangents.

Figure 3.1. NU-METRIC NC-97 Detector

Speed data collection was conducted using the NC-97 detector shown in Figure 3.1. The detector measures 6.5 inches by 5.5 inches and is 0.625 inches thick. The detector is a vehicle magnetic imaging traffic counter that combines the Earth's magnetic field and a vehicle's magnetic mass to measure vehicle speed and length (*11*). The detectors were programmed using serial (RS-232) communications from a personal computer. The NU-METRIC Traffic Flow Analysis (TFA) software was used to program and extract data from the detectors. The software utilized a standard dBase III format to organize the large amount of traffic data collected. Individual vehicle speed and length, along with the time of detection were recorded using this software. In addition, the detector reports surface temperature and wet/dry road conditions.

Previous research at the University of Nebraska-Lincoln has shown that the NU-METRICS NC-97 has an acceptable level of accuracy for use in this study (*12*). The study compared mean speed data collected from an NC-97 detector and an Autoscope 2003 Video Image Analysis System. There was no significant difference in the mean speeds at the 95-percent confidence level. Another study compared the accuracy and visibility of 6 types of speed collection devices (*13*). The devices were pneumatic tubes, magnetic sensors, human observers, radar, tapeswitches, and lidar. The results of this study showed that magnetic sensors (like the NC-97) were very accurate at low speeds, but less accurate as speed increased when compared to the Lateral Acceleration Sensor System (LASS) developed by the Federal Highway Administration (FHWA). The data collection devices' visibility and effect on driver behavior was also studied. The magnetic sensor caused drivers to display brakelights less than one-percent of the time. In comparison, drivers displayed brakelights 1.5-percent of the time for tapeswitches and 5.5-percent of the time for pneumatic tubes. Overall, the NC-97 detectors were chosen for use in this study because they provide sufficient accuracy and are less visible than alternative devices.

Detector Location

The detectors were placed at incremental distances from the stop line to produce an accurate profile of vehicle speeds as they relate to distance from a stop. In general, all locations followed the same pattern of short distances between detectors near the intersection and increasing increments as distance from the intersection increased. For a location using 10 detectors, a possible layout would include detectors at 100, 200, 300, 400, 500, 750, 1000, 1500, 2000, and 2500 feet from the stop line. Since the highest deceleration rates occur near the intersection, more detectors at these locations would result in a more accurate speed profile. Some modifications to this pattern were made on site when influential factors were observed. For example, when rumble bars were present, detectors were placed within 100 feet upstream and downstream of the rumble bars. Detector locations were recorded and are displayed on drawings in Appendix A. The stop line was defined as the line perpendicular to the traveled way passing through the stop sign. The distance from this point to the inside edge of the intersecting highway was also recorded. Distance from the stop line was defined as the distance along the outside edge of the study highway from the stop line to the detector or other object. All measurements were found using a distance-measuring wheel as shown in Figure 3.2.

Figure 3.2. Method of Distance Measurement from Stop on Approach Roadway

Detector Installation

The NU-METRIC NC-97 detectors were installed in the center of the study lane under a polyurethane cover. The protective cover is approximately 15 inches square with a depth of 2 inches. Influence of a driver's operating speed due to detection of the device was assumed to be minimal because the flat, black cover is relatively inconspicuous as indicated in Figure 3.3.

Figure 3.3. NC-97 Detector Under a Protective Cover at a Study Site

The detector and protective cover were secured to the roadway using 4" x ¼" metal screws on concrete surfaces and 6" x ¼" metal screws on asphalt surfaces. Although, the protective cover allowed the detector to withstand vehicular tire impact, the detectors were placed in the center of the lane to minimize such occurrences. The placement of the detector under a protective cover and securing the detector and cover to the roadway are shown in Figures 3.4 and 3.5, respectively.

Figure 3.4. Placement of NC-97 Detector Under a Protective Cover

Figure 3.5. Securing the Protective Cover Over the NC-97 Detector

Chapter 4
DATA ANALYSIS

Data Reduction

The data reduction process involved translating the raw data from the NU-METRIC detector into vehicle speed profiles for each site. After each study was completed, the NU-Metric detector database files were downloaded using Traffic Analyzer Software into a database file. These files were converted into a Microsoft Excel file for easier manipulation. As shown in Figures 4.1 and 4.2, the detector data contents included individual vehicle length and speed, the time offset of detection from the start of the study, the headway (in seconds), between vehicles, and a description of the pavement temperature and surface condition (wet = 0 and dry = 1).

	A	B	C	D	E
1	SPEED	LENGTH	DELAY	HEADWAY	OFFSET
2	62	10	0	46	286
3	21	17	0	1065	1351
4	67	17	0	8	1359
5	63	21	0	195	1554
6	64	21	0	165	1719
7	55	13	0	127	1846
8	59	10	0	260	2106
9	65	12	0	125	2231
10	65	10	0	15	2246
11	63	10	0	31	2277
12	70	21	0	408	2685
13	61	42	0	311	2996
14	24	17	0	16	3012
15	64	21	0	37	3049

Figure 4.1. Vehicle Data

	A	B	C	D	E	F
1	TEMP	WET	OFFSET			
2	64	1	0			
3	64	1	240			
4	66	1	1320			
5	66	1	1500			
6	66	1	1680			
7	66	1	1800			
8	67	1	1800			
9	67	1	2100			
10	67	1	2220			
11	67	1	2640			
12	67	1	2940			
13	67	1	3000			
14	67	1	3120			
15	68	1	3420			

Figure 4.2. Surface Descriptor Data

Once the data were formatted into a spreadsheet, the individual vehicle data was time stamped with the day, hour, minute, and second of occurrence based on the time offset at the beginning of the study. Outliers from the study were removed based on vehicle speeds and lengths. A speed outlier was determined by values that were greater than 3 times the difference between the 25th-percentile and 75th-percentile speeds away from the mean. Speeds outside of this range were removed from the data set. Data for vehicle lengths shorter than 5 ft or longer than 81 ft were also removed. Outlier determination was based upon previous experience with the NU-METRIC detectors (*12*) and suggestions from the NU-METRIC instruction manual (*11*).

Next, the data were sorted according to surface condition (wet/dry), time of day (day/night) and vehicle type (passenger car/heavy vehicle). The first category was determined using the surface description downloaded during each study. There were no wet condition data collected throughout this study. The second category was based on the time of sunrise and sunset as determined by the daily almanac from an Internet website (*14*). Daytime vehicles were determined to be those that occurred from sunrise to sunset and nighttime vehicles were determined to be those that occurred from sunset to sunrise. The final separation was based on vehicle length. Vehicles with an axle spacing of less than 22 ft were considered passenger cars and vehicles with an axle spacing of greater than or equal to 22 ft were considered heavy vehicles. Each step was repeated for each detector at each study site.

After sorting the data into categories, descriptive speed statistics were calculated for each detector location. The speed statistics calculated were mean speed, standard deviation, 95th-percentile speed, 85th-percentile speed, 15th-percentile speed, and 5th-percentile speed. The individual detector location statistical summaries were combined to create a speed profile for each category at each location. Statistical summaries for each site are shown in Appendix A.

Model Development

The initial objective of this research was to find a single speed profile model that would be appropriate for all vehicles on horizontal curves approaching a stop-controlled intersection. The procedure used to determine such a model included the following steps:

Step 1: Determine which data sets would give valid results.

Step 2: Determine which regression model would provide a significant relationship between speed and distance from the stop.

Step 3: Determine possible influential factors and test their significance using comparison of regression lines.

Step 1. Data Sets Used

Since the majority of the study sites were located in rural Nebraska, the average daily traffic (ADT) was usually very low (see Table 2.4). The sample size of each vehicle type, time of day, and site are reported in Appendix A. Because of the small sample sizes collected during this study, this report will focus on speed data from all vehicles, daytime passenger cars, and daytime heavy vehicles. All of the data considered were collected during dry weather conditions as reported by the NU-METRIC detector.

Step 2. Model Selection
The model that was chosen to represent the speed profile was the multiplicative model, also known as the power function. Equation 4.1 defines the multiplicative model in exponential form, while Equation 4.2 defines its linear form.

$$Y = a\,X^{b} \tag{4.1}$$

where,

Y = dependent variable
X = independent variable
a = coefficient of X
b = exponent of X

$$LnY = A + bLnX \tag{4.2}$$

where,

Y = dependent variable
X = independent variable
A = intercept of linear model
b = slope of linear model

The multiplicative model provided a statistically significant relationship between speed and distance at the 99-percent confidence level. The model was chosen because it was relatively easy to use and related to the general assumptions of this research. With the multiplicative model, the assumption that the speed at the stop line is zero holds true and the non-linear characteristic of the speed profile is duplicated. The multiplicative model assumes that the speed data recorded at one detector is independent of the speed recorded at the previous or subsequent detectors. Since the detectors were set up in a series, this may not be the case. The layout of the detectors may cause a serial correlation between the speeds recorded by the progression of detectors. However, since the models developed are to be used to predict speeds in cases where similar serial correlation can be expected, this condition is acceptable.

Step 3. Comparison of Regression Lines
Possible factors influencing vehicle speed were determined during data collection and analysis. They were found to be: presence of horizontal curvature, vehicle type, curve type, posted speed, median type, rumble bars presence, surface condition, and degree of rutting. Each factor was tested for significance by comparing intercepts and/or slopes of regression lines. STATGRAPHICS Plus 5.0 (*15*) was used to conduct the comparison of regression lines for this research. The input required a simple regression model of the form Y = a + bX and a categorical variable. For each comparison in this research, the independent variable, X, was distance from the stop line and the dependent variable, Y, was the 95th-percentile speed. The 95th-percentile speed was chosen as the dependent variable because it is assumed to closely represent the design speed of the roadway (*4*). The categorical variable corresponds to the possible influential factors discussed earlier in this section. The statistical software groups the speed and distance data by the categorical level. The regression line developed by each categorical level is then compared to each other. For example, when the significance of curve type is being

tested, the categorical variable is curve type and the levels are single, reverse, compound, and none. The goal of this comparison is to test to see if a single model can be used across categories (*15*), in other words, to test whether a single model can be used at all sites, uninfluenced by roadway characteristics.

The statistical test used for this analysis was a conditional sums of squares. This report includes an analysis of variance for the intercept and slope of the model, which determines whether the intercepts and/or the slopes differ among the levels of the categorical variable. The null hypothesis for each test was that there were no statistically significant differences between the slopes or the intercepts in the regression lines. The alternative hypothesis was that there were statistically significant differences in the slopes or intercepts. A 95-percent level of confidence was assumed for the comparison of regression lines. If a comparison had no significant difference, it was removed from the list of possible factors. If a comparison had a significant difference, further investigation was performed to understand the cause of the difference.

Some additional statistical tools were used to determine the validity of the models being tested. The adjusted coefficient of determination, R_a^2, was used to measure the proportion of variability in the model for the dependent variables. Scatter plots were used to visually inspect the difference between regression lines. Lastly, the statistical software reported regression equations for each regression line that was also compared to see the actual difference in slope and intercept.

Chapter 5
RESULTS

Search for a Representative Speed Profile Model

The linear form of the multiplicative model relates the natural log of the dependent variable of 95th-percentile speed to the natural log of the independent variable of distance. To develop a final model that would be appropriate for all vehicles and all locations, the significance of each needed to be tested. The test used was the comparison of regression lines. The categorical variables and corresponding levels used are shown in Table 5.1. A confidence level of 95 percent ($\alpha = 0.05$) was used to determine significance for all comparisons.

Table 5.1. Comparison of Regression Categories Tested

Categorical Variables	Levels
Vehicle Type	all, daytime passenger car, daytime heavy vehicle
Curve Type	single, reverse, compound, none
Horizontal Curvature	horizontal curve, none
Posted Speed	55 mph, 60 mph
Median	raised/painted, none
Rumble Bars	present, none
Roadway Surface	very good, good, fair
Rutting	< 6 mm, 6 mm thru 13 mm

Since aggregate data was used as opposed to individual speed data, the total variability in speed and nature of the variability may be reduced (*16*). The R_a^2 values may overstate the actual variation in 95th-percentile speed described by distance. Other factors, outside of the scope of this research, may also influence a driver's choice for speed at a given distance from the stop line. Examples such as driver age, previous knowledge of the roadway, signage, and vertical alignment were not included in the analysis.

Vehicle Type

The purpose of the initial comparison was to test whether passenger car daytime data was a good representation of the entire vehicle population and whether there was a significant difference between passenger cars and heavy vehicles. The three data samples compared were 1) all vehicle data, 2) daytime passenger car data, and 3) daytime heavy vehicle data. The result of this comparison (Comparison 1, Appendix B) was that there was a significant difference between the three speed samples. All of the regression lines and statistical summaries can be found in Appendix B. The comparisons are numbered and will hereby be referenced for convenience by Comparison Number, Appendix B.

Since the previous comparison resulted in significant differences, the entries were divided to further investigate the differences in vehicle population. First, all vehicle data were compared to daytime passenger car data and second, daytime passenger car data were compared to daytime heavy vehicle data. The results indicated no significant differences between all vehicles and daytime passenger cars (Comparison 2, Appendix B). This result was not unexpected since the majority of the vehicles sampled were passenger cars. In the second comparison, it was found that there were significant differences between passenger cars and heavy vehicles (Comparison 3, Appendix B). As a result of this finding, separate speed profile equations were developed for both types of vehicles. The results of the comparisons are shown in Table 5.2.

Table 5.2. Comparisons for Vehicle Type

Comparison Entries	Significant Difference Slope	Intercept
All Vehicles/Day PC/Day HV[a]	Yes	Yes
All Vehicles/Day PC[b]	No	No
Day PC/Day HV[c]	Yes	Yes

a: Comparison 1, Appendix B b: Comparison 2, Appendix B c: Comparison 3, Appendix B

Passenger Cars Comparisons

For each vehicle type, a comparison was made to determine if the type of curve had an effect on the vehicle speed profile. For passenger cars, there was no statistically significant difference between the four types of curves studied at the 95-percent confidence level (Comparison 4, Appendix B). Since the regression line for the no curve data had a higher slope and a lower intercept than the remaining three curve types, it was decided to test the differences between the sites with a horizontal curve and the sites without a horizontal curve. The same result was obtained from this comparison. There was no significant difference between the two categories (Comparison 5, Appendix B). Based on these results, shown in Table 5.3, the data from all of the sites, regardless of curve type, were included in the regression model. Regression line comparisons were then conducted to determine the effect of posted speed, surface condition, degree of rutting, median type, and presence of rumble bars. There were no significant differences in intercept or slope for the regression lines at the 95 percent confidence level (Comparisons 6 through 10 in Appendix B). These results are summarized in Table 5.3. For daytime passenger car data, the influential site characteristics had no significant effect.

Table 5.3. Comparisons for Daytime Passenger Car

Comparison Entries	Significant Difference Slope	Intercept
PC - Simple/Reverse/Compound/No Curve[a]	No	No
PC - Curve/No Curve[b]	No	No
PC - 60 mph/55 mph[c]	No	No
PC - Median/No Median[d]	No	No
PC - Rumble Bars/No Rumble Bars[e]	No	No
PC - Surface Condition - Very Good/Good/Fair[f]	No	No
PC - Rutting - Good/Fair[g]	No	No

a: Comparison 4, Appendix B
b: Comparison 5, Appendix B
c: Comparison 6, Appendix B
d: Comparison 7, Appendix B
e: Comparison 8, Appendix B
f: Comparison 9, Appendix B
g: Comparison 10, Appendix B

Final Speed Profile Model for Passenger Cars

For daytime passenger cars in dry conditions, the comparison of regression lines indicated that none of the factors had a significant effect on the speed profile. The final model, developed using simple regression, included the data from all 15 study sites for daytime passenger cars. Distance was the independent variable and 95th-percentile speed was the dependent variable. The relationship, expressed by Equation 5.1, had an R_a^2 of 89.6 percent and a correlation coefficient of 0.947. The relationship was also statistically significant at the 95-percent confidence level. The speed profile is shown in Figure 5.1.

$$V_{95} = 10.42\ D^{\,0.250} \qquad \textbf{(5.1)}$$

where,

V_{95} = 95th-percentile speed of daytime passenger cars in dry conditions, mph, and

D = Distance from stop line, ft.

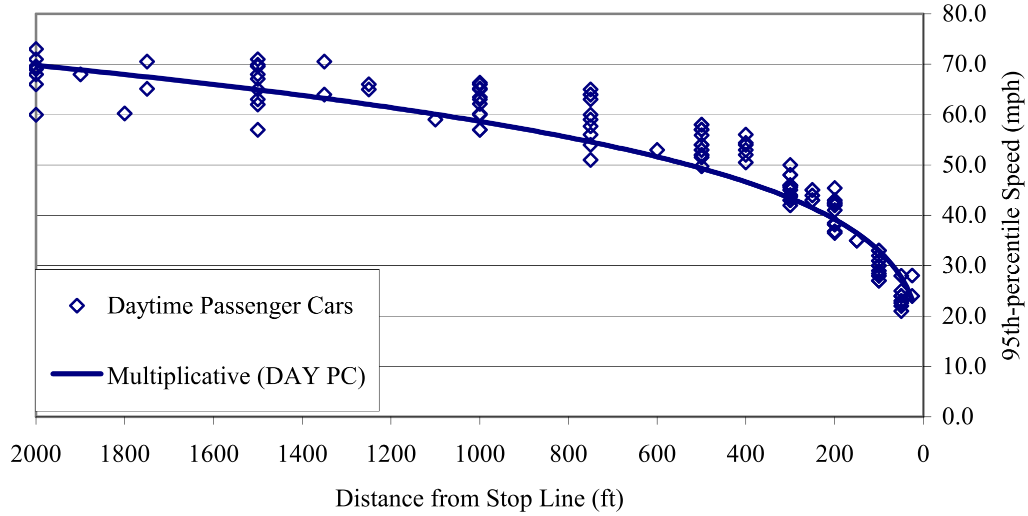

Figure 5.1. Speed Profile for Daytime Passenger Cars in Dry Conditions (Equation 5.1)

Heavy Vehicle Comparisons

The comparison tests of the site characteristics were conducted in the same order for heavy vehicles as for passenger cars. Initially, it was determined whether the curve type had an effect on the speed profile. The results of this comparison, shown in Table 5.4, revealed that there was no significant difference in the slope of the regression lines, but there was a significant difference in the intercept of the regression lines (Comparison 11, Appendix B). Because of this, the data were investigated further to find where the difference occurred.

Table 5.4. Comparisons for Daytime Heavy Vehicle Curve Type

	Significant Difference	
Comparison Entries	**Slope**	**Intercept**
HV - Simple/Reverse/Compound/No Curve[a]	No	Yes
HV - Simple/Reverse/Compound[b]	No	No
HV - Curve/No Curve[c]	No	Yes

a: Comparison 11, Appendix B
b: Comparison 12, Appendix B
c: Comparison 13, Appendix B

In order to do this, two comparisons were conducted. First, the three curve types were compared to one another then the curve data were compared to the no-curve data. The result of the comparison between the three curve types was that there was no significant difference in the slope or the intercept. (Comparison 12, Appendix B). The result of the comparison between curve sites and no curve sites was that there was a significant difference between the intercepts (Comparison 13, Appendix B). Both results are summarized in Table 5.4. The intercept of the no-curve data was significantly greater than the intercept of the curve data. This means that the speed profile of heavy vehicles on the sites without curves was higher than that on approaches with curves. Since there was a significant difference, it was decided to separate the site data based on whether it contained a horizontal curve for further study of site characteristics and model development.

The data from the 12 sites that contain a horizontal curve were used to evaluate the effect of the site characteristics. As shown in Table 5.5, there were no significant differences in intercept or slope for the regression lines categorized by posted speed, presence of a median or rumble bars, surface condition, or rutting at the 95-percent confidence level (Comparison 14 through 18 in Appendix B). Once again, the different characteristics did not significantly influence the speed profile based on daytime heavy vehicles at sites with horizontal curves approaching a stop.

Table 5.5. Comparisons for Daytime HV and Horizontal Curvature

	Significant Difference	
Comparison Entries	**Slope**	**Intercept**
HV Curve - 60 mph/55 mph[a]	No	No
HV Curve - Median/No Median[b]	No	No
HV Curve - Rumble Bars/No Rumble Bars[c]	No	No
HV Curve - Surface Condition - Very Good/Good/Fair[d]	No	No
HV Curve - Rutting - Good/Fair[e]	No	No

a: Comparison 14, Appendix B
b: Comparison 15, Appendix B
c: Comparison 16, Appendix B
d: Comparison 17, Appendix B
e: Comparison 18, Appendix B

The data from the three sites without horizontal curves were also studied and a speed profile was developed. Similar comparisons were conducted for these sites. The results, shown in Table 5.6, indicate that there were no significant differences in intercept or slope for the regression lines categorized by posted speed, presence of a median or rumble bars, or rutting at the 95-percent confidence level (Comparison 19 through 21 in

Appendix B). Since all three sites had the same surface rating of very good, a comparison could not be completed. The final result of the comparison of regression lines for this data was that there were no significant differences caused by site characteristics.

Table 5.6. Comparisons for Heavy Vehicles and No Horizontal Curvature

	Significant Difference	
Comparison Entries	**Slope**	**Intercept**
HV No Curve - 60 mph/55 mph[a]	No	No
HV No Curve - Median and Rumble Bars/None[b]	No	No
HV No Curve - Rutting - Good/Fair[c]	No	No

a: Comparison 19, Appendix B
b: Comparison 20, Appendix B
c: Comparison 21, Appendix B

Final Speed Profile Model for Heavy Vehicles at Curve Sites

The following final model for daytime heavy vehicles in dry conditions was developed using the data from the 12 sites with horizontal curves. Distance was the independent variable and 95th-percentile speed was the dependent variable for the regression analysis. The relationship, expressed by Equation 5.2, had an R_a^2 of 84.4 percent and a correlation coefficient of 0.919. The relationship was also statistically significant at the 95-percent confidence level. The speed profile is shown in Figure 5.2.

$$V_{95} = 12.0\ D^{\,0.219} \qquad \textbf{(5.2)}$$

where,

V_{95} = 95th-percentile speed of daytime heavy vehicles on curved alignments in dry conditions, mph, and

D = Distance from stop line, ft.

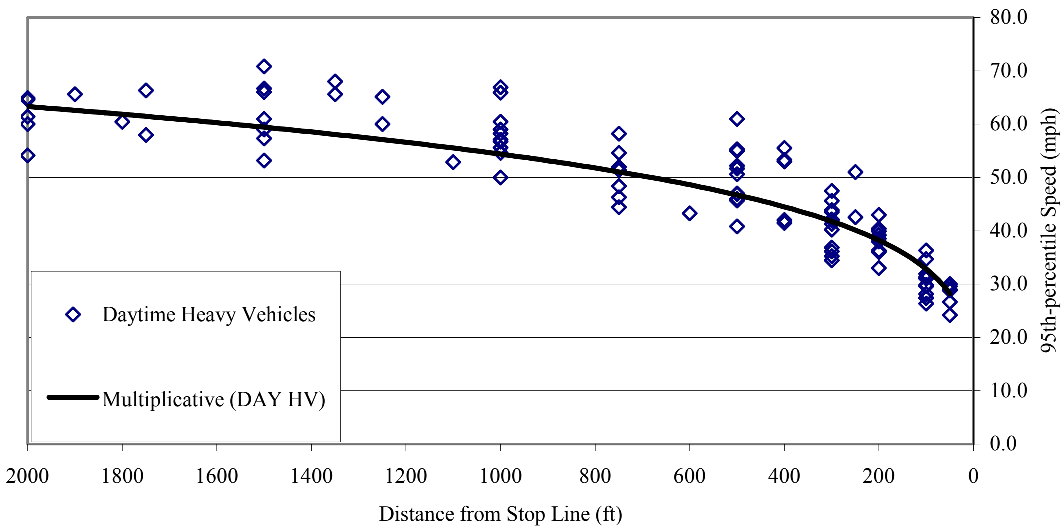

Figure 5.2. Speed Profile for Daytime Heavy Vehicles on Curved Alignments in Dry Conditions (Equation 5.2)

Final Speed Profile Model for Heavy Vehicles at Tangent Sites

The following final model for daytime heavy vehicles in dry conditions was developed using the data from the 3 sites without horizontal curves. Distance was the independent variable and 95th-percentile speed was the dependent variable for the regression analysis. The relationship, expressed in Equation 5.3, had an R_a^2 of 77.7 percent and a correlation coefficient of 0.885. The relationship was also statistically significant at the 95-percent confidence level. The speed profile is shown in Figure 5.3.

$$V_{95} = 14.6\, D^{\,0.197} \qquad \textbf{(5.3)}$$

where,

V_{95} = 95th-percentile speed of heavy vehicles on tangent alignments in dry conditions, mph, and

D = Distance from stop line, ft.

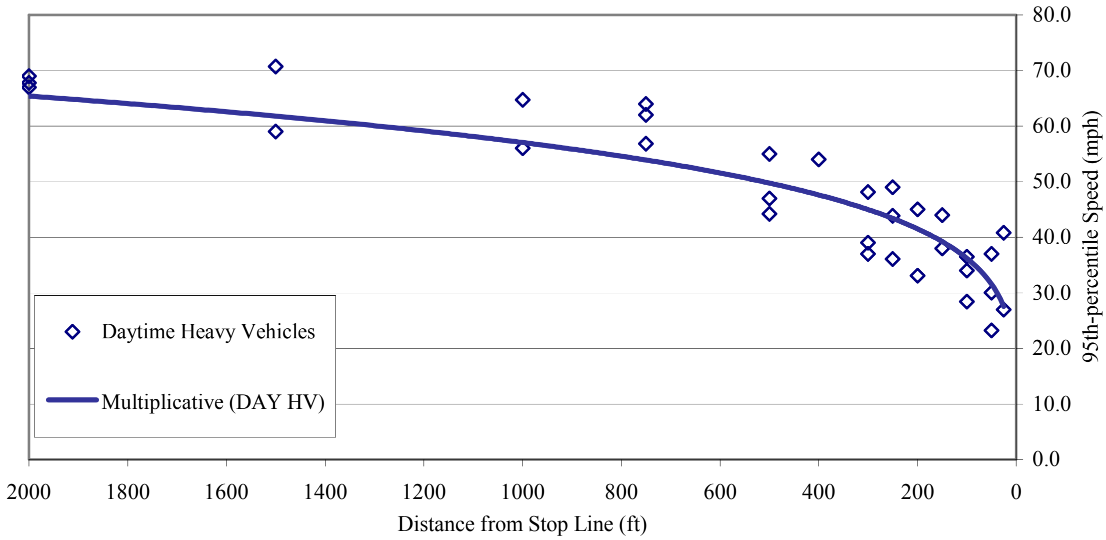

Figure 5.3. Speed Profile for Daytime Heavy Vehicles on Tangent Alignments in Dry Conditions (Equation 5.3)

Chapter 6
CONCLUSIONS RELATING TO SPEED PROFILE MODELS

Conclusions

Speed profiles for vehicles decelerating to a stop on rural two-lane two-way highways that have a horizontal curve on the intersection approach were developed using data from 15 study sites in Nebraska. The multiplicative model was used to provide a prediction equation of the speed profile. Separate profiles were created for passenger cars and heavy vehicles because the regression lines were significantly different. The heavy vehicle data was separated further for alignments with and without a horizontal curve. It was concluded that posted speed, 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-percent confidence level. The regression equations for the three models developed are summarized in Table 6.1.

Table 6.1. Regression Equations for Final Models

Vehicle Type	Approach Type	Developed Equation
Daytime Passenger Car	All	95th-Percentile Speed = 10.4 Distance$^{0.250}$
Daytime Heavy Vehicle	Curve	95th-Percentile Speed = 12.0 Distance$^{0.219}$
Daytime Heavy Vehicle	No Curve	95th-Percentile Speed = 14.6 Distance$^{0.197}$

95^{th}-Percentile Speed (mph) & Distance (feet)

The multiplicative model format provided a statistically significant relationship between distance and 95^{th}-percentile speed for the three speed profiles that were developed from this research. The plots of the multiplicative equations are shown in Figure 6.1. The curves show that passenger cars generally have a greater free-flow speed at the approach to the curve and subsequent intersection than heavy vehicles. The curves also show that the passenger cars decelerate at a greater rate than the heavy vehicles. These conclusions are further reinforced by the fact that the exponent of the regression line for passenger cars is greater and the coefficient is lower than those in both heavy vehicle equations. All three curves merge together near the stop line. The speed profiles developed can be used to predict 95^{th}-percentile speeds of vehicles as they approach a stop on sections with or without a horizontal curve.

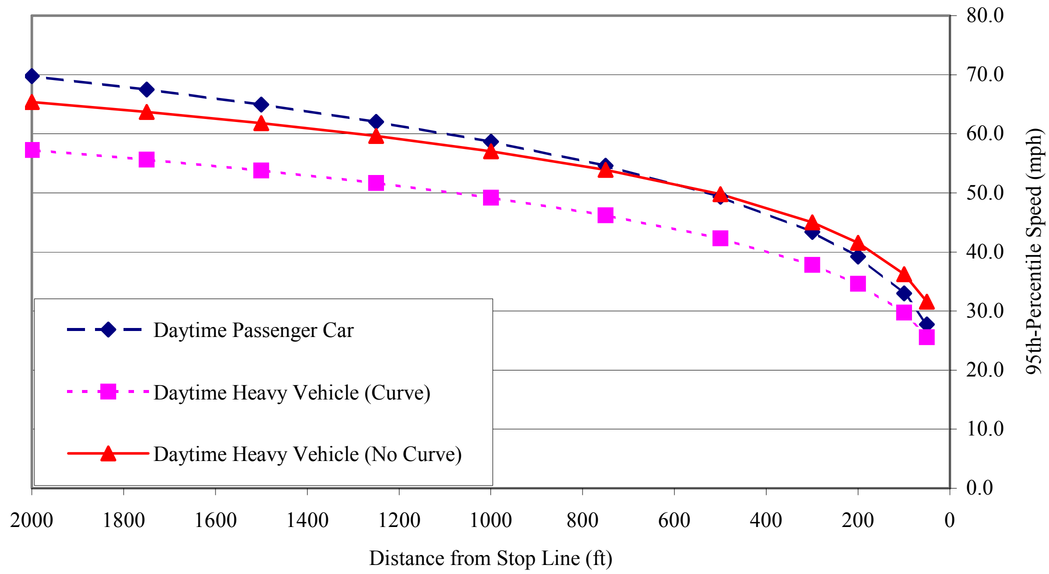

Figure 6.1. Vehicle Speed Profiles in Dry Conditions Developed from Research Data

The models were evaluated based on the NDOR and AASHTO design policies and the AASHTO deceleration rates. The AASHTO design policies (*4*) recommended that the design speed of the horizontal curve closely represent the 95th-percentile speed of vehicles. The speed profiles developed in this research are based on this recommendation. With that in mind, the speed profiles are compared to the design policies currently used in the state of Nebraska. The NDOR Roadway Design Manual (*1*) stated that the posted speed is generally from 5 to 10 mph less than the design speed. This would result in a design speed ranging from 60 – 70 mph for the sites studied. From the speed profiles in Figure 6.1, it can be seen that the approach speeds of passenger cars at a distance of 2000 feet from the stop line, are within this range, whereas heavy vehicles are slightly below this range. For the design of horizontal curves, the NDOR design manual recommended choosing design speeds that are greater than 30 mph and less than the design speed minus 20 mph. For the sites studied, the range of design speed would be 50 to 30 mph. When compared to the speed profiles, this coincides with the 95th-percentile speed of vehicles within 500 ft of the stop line. The NDOR policy fits the speed profile of vehicles near the intersection and in free-flow conditions. The NDOR policy doesn't coincide with the speed profile for horizontal curves that are designed between 500 and 2000 feet from the stop line. At these distances, the free-flow design speed is too high and the design speed for horizontal curves near an intersection is too low.

The speed profile determined for passenger cars in this project is compared to the 2001 Green Book deceleration distances for passenger car vehicles approaching intersections in Figure 6.2. Since there were no significant differences found between approaches with horizontal curves and those without, the comparison is assumed to be valid. From the 2001 Green Book, Curve E relates to a comfortable deceleration rate reaching a final speed of zero and Curve X relates to the minimum braking distance or maximum deceleration on dry pavement for passenger cars approaching intersections (*4*). The results from this study show that the 95th-percentile speed profile of passenger cars approaching a stop is more gradual than indicated by the AASHTO curves. The prediction model approaches the comfortable rate curve about 200 to 300 ft from the stop line and the maximum deceleration curve about 50 ft from the stop line. Only in the last 50 ft of the approach does the prediction model resemble the shape of the AASHTO curves. The use of the AASHTO curves to relate distance to speed would result in speeds that are too high when compared to the actual 95th-percentile speed of traffic found by this research.

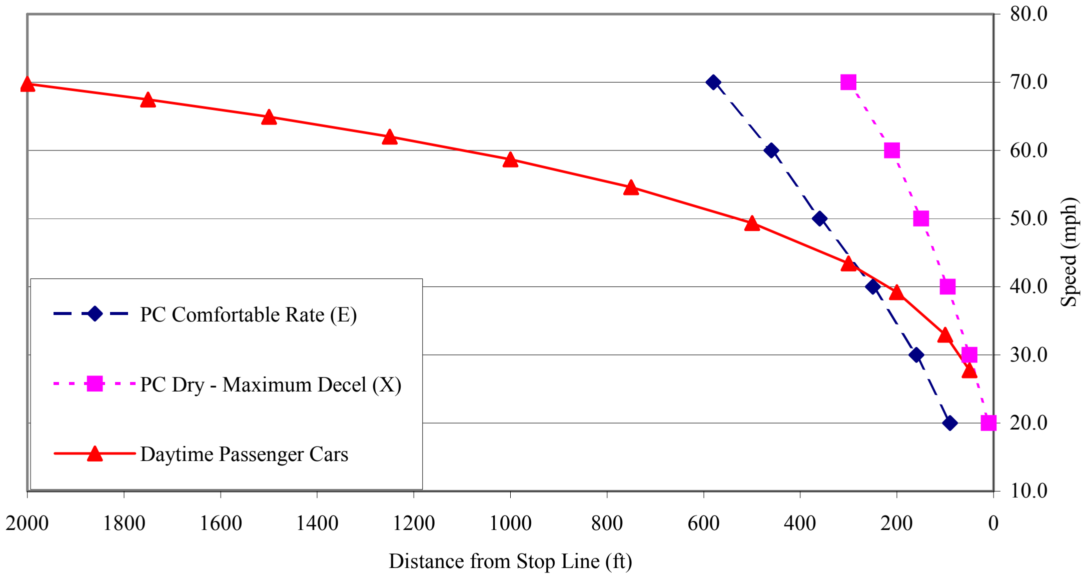

Figure 6.2. 2001 Green Book Deceleration Curves versus Developed Speed Profile for Daytime Passenger Cars in Dry Conditions
Source: (*4*)

During the comparison of regression lines, 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 curves in Figure 6.3 show that the speed of heavy vehicles on non-curve approaches was generally about 8 mph higher than on sites that exhibited horizontal curvature. The shapes of the speed profiles remain nearly parallel throughout the deceleration process since there was no significant difference in the slopes of the lines. This means that the rate of deceleration remains nearly the same on all approaches to intersections, except near the stop line.

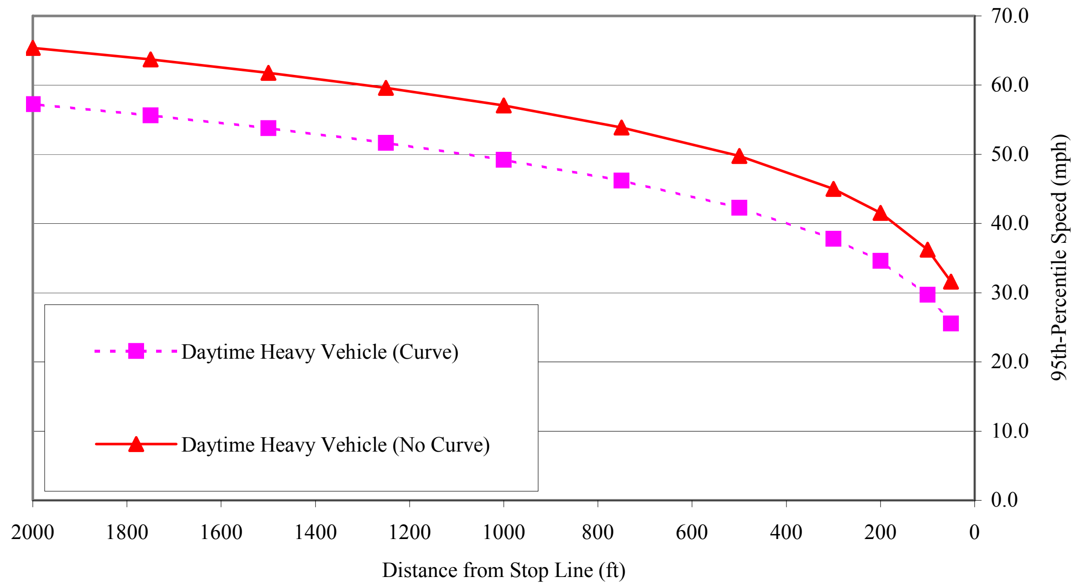

Figure 6.3. Speed Profiles for Heavy Vehicles

A major safety issue for heavy vehicles on horizontal curves is the chance of overturning. A rollover is caused by lateral acceleration on the vehicle produced by the roadway curvature, superelevation rate, side friction between the tires and the roadway surface and speed of the vehicle. The speed at which overturning will occur is also affected by the vehicle type and loading condition (*17*). A reduction in speed will result in a reduction in the chance for a heavy vehicle to overturn. This fact may explain why heavy vehicles reduced their speed at the sites that contained a horizontal curve prior to the stop. The method of data collection in this research did not lend itself to determine whether heavy vehicle drivers were familiar with the roadway alignment.

This research was limited to two-lane two-way rural highways in Nebraska. The posted speed limit at the sites was either 55 mph or 60 mph. The research investigated the speed profile within approximately 3000 feet of the stop-controlled intersection.

Chapter 7
DEVELOPMENT OF A PROCEDURE TO DETERMINE THE MINIMUM RADIUS CURVE APPROPRIATE FOR A TWO-LANE TWO-WAY ROADWAY ALIGNMENT APPROACHING A STOP CONDITION

Background

One objective of this research was to develop models that describe the operating speed profiles of vehicles traversing horizontal curves on approaches to stop-controlled intersections on rural two-lane two-way highways. These models can predict the 95th-percentile operating speed of a vehicle at a given distance from the stop line. Since such models have now been determined and conclusions drawn about driver deceleration behavior from those models, a procedure for the design of horizontal curves on rural highways that must accommodate vehicles transitioning from high speeds to a stop-controlled intersection can be developed.

General Guidelines for Alignments with Horizontal Curvature in Advance of a Stop

Driver accounts of accidents occuring at the study site locations indicated that many accidents that occurred due to roadway horizontal curvature near a stop were the result of the driver not being aware that:

1. The roadway alignment had a horizontal curve, and
2. There was a stop condition along the roadway ahead.

The task of following the roadway alignment in a vehicle involves tracking the lane path by the driver. This action requires the vehicle operator to visually evaluate the path ahead, predict the steering and speed control inputs necessary for sustaining the desired path, make the control inputs, then using visual feedback, operate the controls to compensate for deviations. The tracking process continues until the vehicle reaches the driver's destination or comes to a stop. This task is relatively simple for an experienced driver if the roadway is free of traffic and obstacles and if the driver's expectations are met by the roadway design (*18*).

According to Bonneson, "the literature review of driver steer behavior indicates that drivers initiate their steer based on their perception of curve location. The break in alignment at the point of curvature (PC) is a key piece of information available to the driver's anticipatory response mechanism. However, this apparent benefit of a tangent-to-curve transition is not generally acknowledged in the field of highway design." (*19*)

A horizontally curved alignment followed by a stop multiplies the workload that a vehicle operator must process to successfully complete the driving task. Therefore, this research recommends the use of a simple curve in the horizontal alignment (without spiral transitions) regardless of the curve radius dimension when a stop condition is in near proximity. The photographs in Figure 7.1 show how prominently the roadway curvature appears when a spiral curve is not included.

Figure 7.1 PC of Curve as Driver Visual-Tracking-Path Cue

Another general safety guideline for horizontally curved alignments approaching a stop is to provide the most sight distance economically feasible in advance of the intersection. Figure 7.2 shows a stop-controlled intersection that is partially obscured by a roadside tree.

Figure 7.2 Intersection with Stop Sign Obscured by Roadside Tree

Approach for Procedure Development

The objective of defining a procedure to determine the minimum radius curve appropriate for a roadway alignment approaching a stop was to assure that:

1. The visual expectations of the driver were met,
2. The comfort of the driver and passengers within the vehicle was optimized,
3. The curve design was a simple curve without spirals,
4. The vehicle speed within the limits of the curve were reasonable,
5. Sufficient braking distance to the stop was available, and
6. Rates of deceleration to a stop were reasonable.

Results of the analysis of vehicle speeds approaching a stop with and without curvature indicated that there were no statistically significant differences at the 95-percent confidence level between the 95th-percentile speed profiles of both alignment types. Therefore, speed profiles were developed from the data collected at the three tangent sites. Vehicle speeds along the tangent sites were considered to model preferred driver deceleration behavior in advance of the stop. In general, tangent alignments have better sight distance, fewer signs and no lateral acceleration. These conditions allow the driver to slow to a stop with very few distractions, creating what is considered by this research to be the driver's "preferred" speed along the deceleration path. Free-flow passenger car speeds were used to develop the speed profiles to be incorporated into the procedure since the results of previous work in this study showed that those 95th-percentile speeds were the highest of all vehicles and heavy vehicle drivers along curved alignments appeared to reduce their speeds in advance of the stop voluntarily to reduce the risk of rollover.

Speed profiles were developed by tracking individual free-flow passenger cars in dry conditions from detector to detector starting from a position 2000 ft in advance of the stop at the intersection. Vehicles at this location were categorized into four "design" speed groups: 55-59 mph, 60-64 mph, 65-69 mph, and 70 mph and greater. This categorization allowed the procedure to evaluate a range of approach speeds. Intuitively, drivers operating their vehicles on tangent sections under free-flow conditions should be traveling at what they consider to be a "preferred" speed for the roadway at this location. Therefore, approach speed was substituted for design speed in the procedure. For example, procedure users would choose the speed profile for 65-69 mph to represent the design speed of their facility if it was 65 mph.

Once the data was separated by approach speed into one of the four categories, regression analysis was performed to determine the best-fit line for speed as a function of distance from a stop-controlled intersection. The results of the analysis are displayed in Table 7.1. Figures 7.3, 7.4, 7.5 and 7.6 show the graphical forms of the equations along with the data points for each speed category and Figure 7.7 shows the graphical forms of all equations in a single figure.

Table 7.1. Speed Profile Regression Analysis Results for each Approach Speed Category

Approach Speed Category (mph)	Regression Equation (y = speed),(x = distance)	R_a^2
55-59	y = 11.829Ln(x) - 30.073	0.8586
60-64	y = 13.155Ln(x) - 36.343	0.8881
65-69	y = 14.124Ln(x) - 40.347	0.8849
>70	y = 15.575Ln(x) - 46.832	0.8788

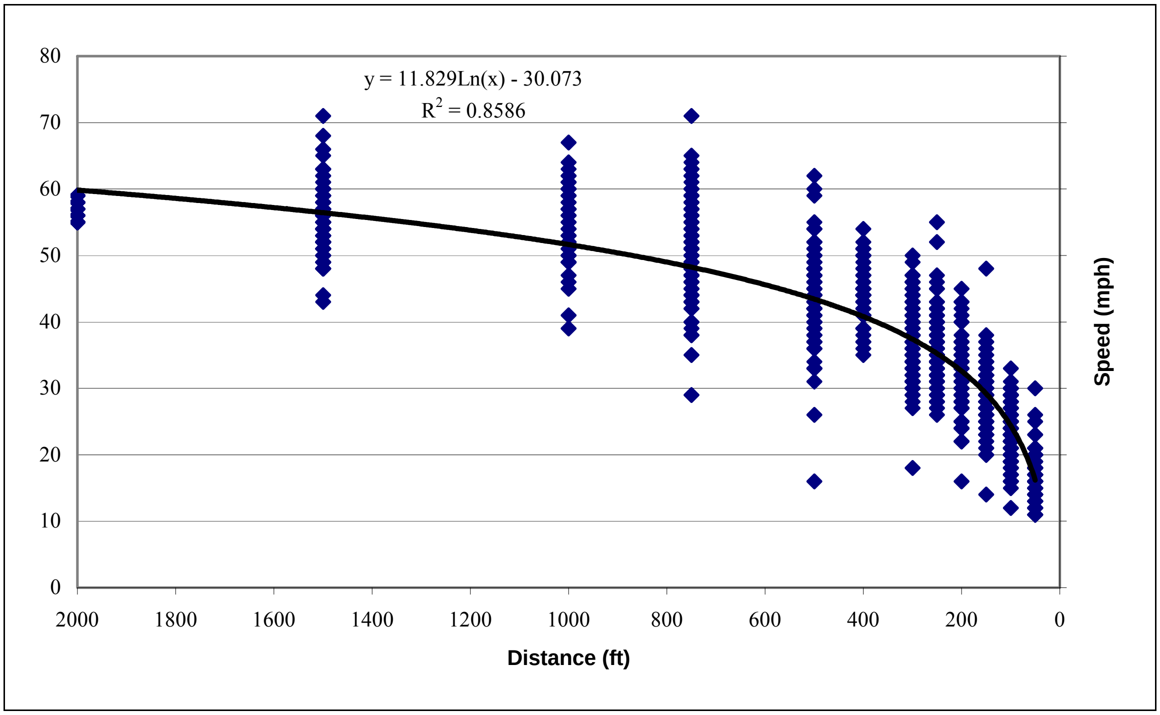

Figure 7.3. Speed Profile of Free-flow Passenger Cars in Dry Conditions Traveling 55-59 mph at 2000 ft from the Stop at Tangent Alignment Sites

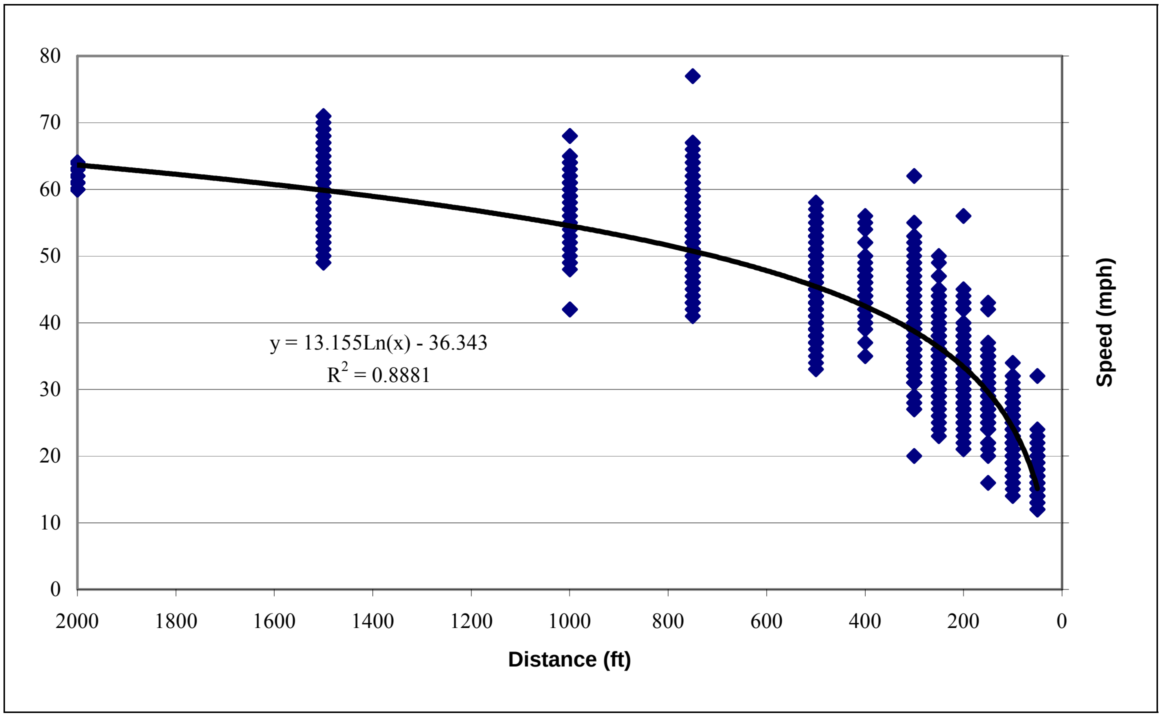

Figure 7.4. Speed Profile of Free-flow Passenger Cars in Dry Conditions Traveling 60-64 mph at 2000 ft from the Stop at Tangent Alignment Sites

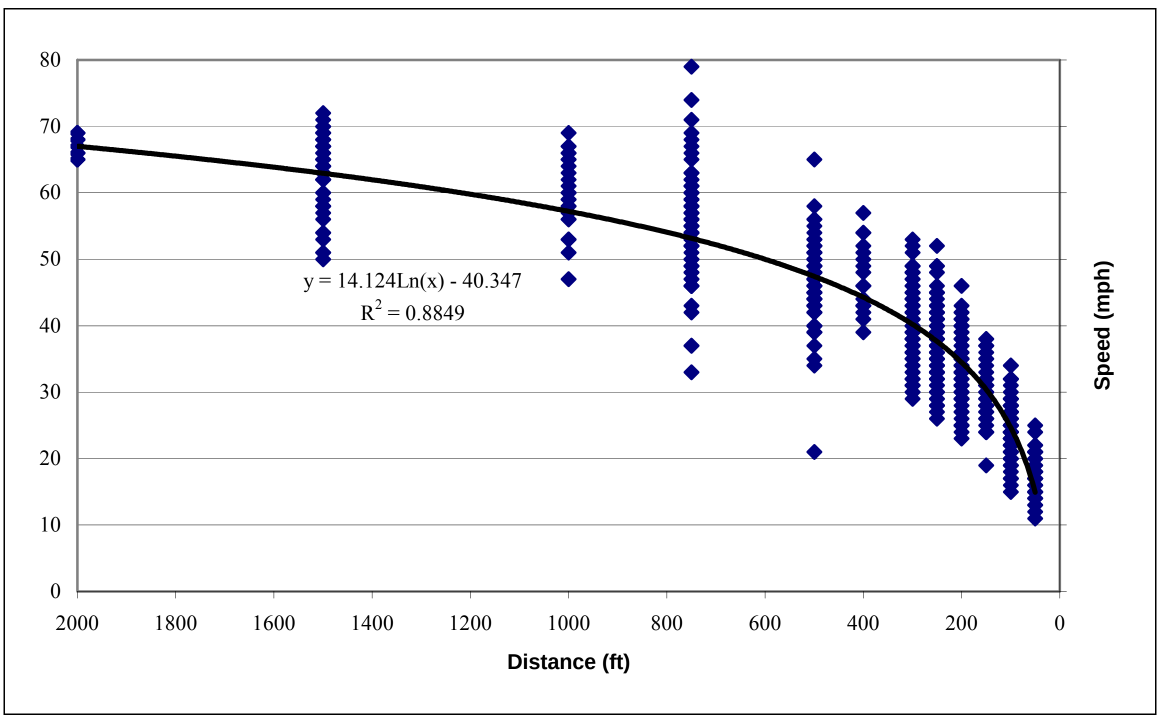

Figure 7.5. Speed Profile of Free-flow Passenger Cars in Dry Conditions Traveling 65-69 mph at 2000 ft from the Stop at Tangent Alignment Sites

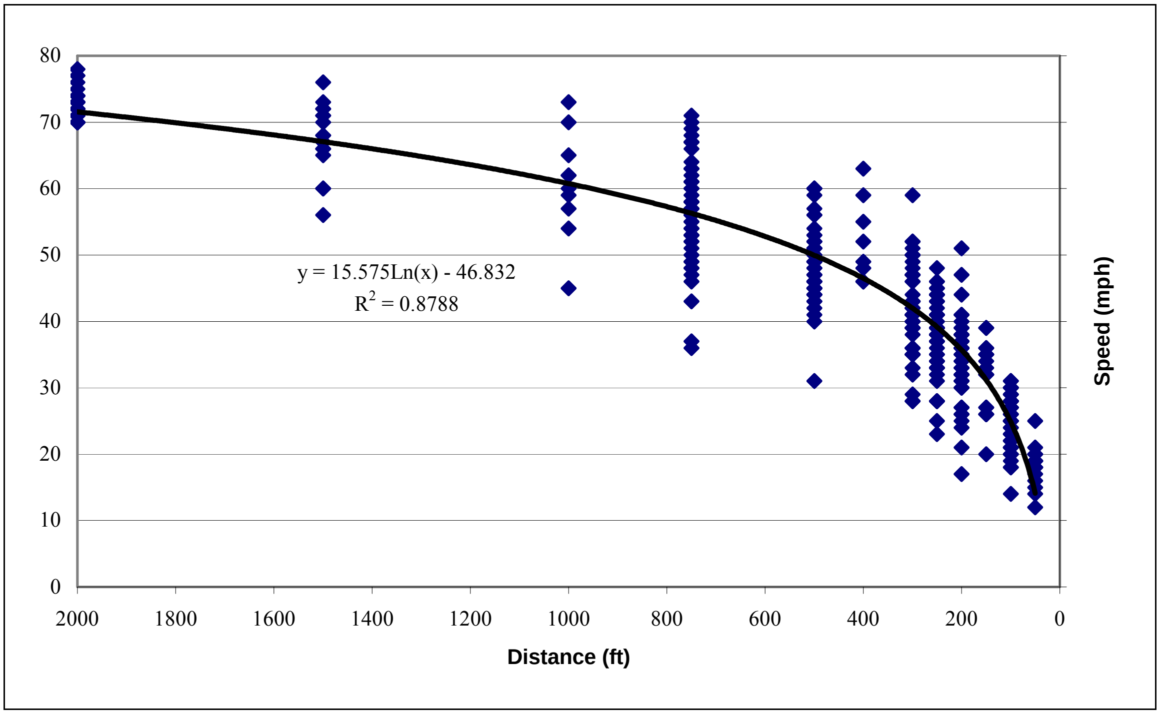

Figure 7.6. Speed Profile of Free-flow Passenger Cars in Dry Conditions Traveling 70 mph and Greater at 2000 ft from the Stop at Tangent Alignment Sites

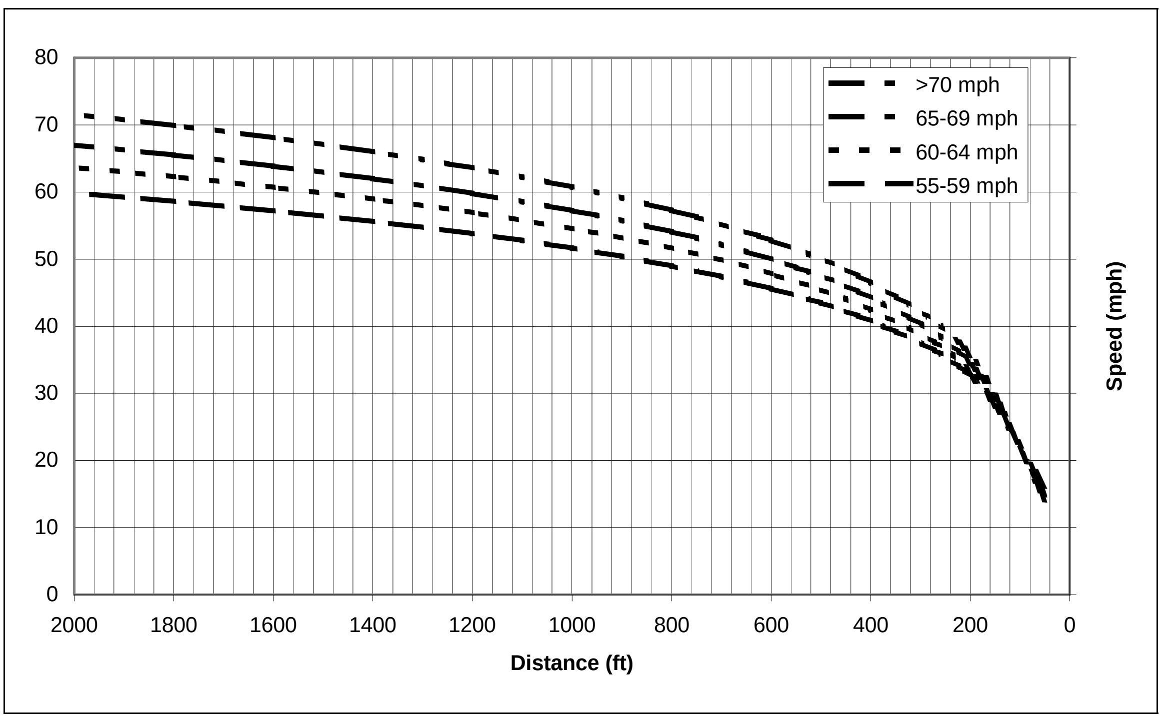

Figure 7.7. Speed Profiles of Four Entry Speed Categories of Free-Flow Passenger Cars in Dry Conditions Approaching a Stop at Tangent Sites

Average Time to Decelerate

The time to decelerate at a given distance from the stop was required in order to calculate the rate of change in lateral acceleration along the curve from PC to PT. A regression analysis was performed to find the best-fit line for deceleration time as a function of distance to a stop-controlled intersection. The analysis was performed using free-flow passenger car speeds from the three tangent sites. Individual vehicles were tracked from detector to detector starting from 2000 feet in advance of the curve to the intersection. Deceleration times were separated into the four approach speed categories mentioned in the previous section. The results of this analysis are displayed in Table 7.2. Figures 7.8, 7.9, 7.10, and 7.11 display the graphical results for the speed categories of 55-59 mph, 60-64 mph, 65-69 mph, and 70 mph and greater respectively. Figure 7.12 displays the deceleration time regression line for each of the speed categories.

A comparison of R_a^2 values indicated that polynomial equations provide a slightly better fit than the linear equations used in this procedure, but the polynomial equations were not used because they produced several counter-intuitive results later in the design process. The difference in time results between the polynomial and linear equations was relatively small. For these reasons, the procedure utilized the linear equations.

Table 7.2. Deceleration Time Regression Analysis Results for each Approach Speed Category

Approach Speed Category (mph)	Regression Equation (y = time, sec),(x = distance, ft)	R_a^2
55-59	y = 0.0162x + 2.798	0.9072
60-64	y = 0.0155x + 2.555	0.8934
65-69	y = 0.0152x + 2.336	0.8777
>70	y = 0.0145x + 2.362	0.8752

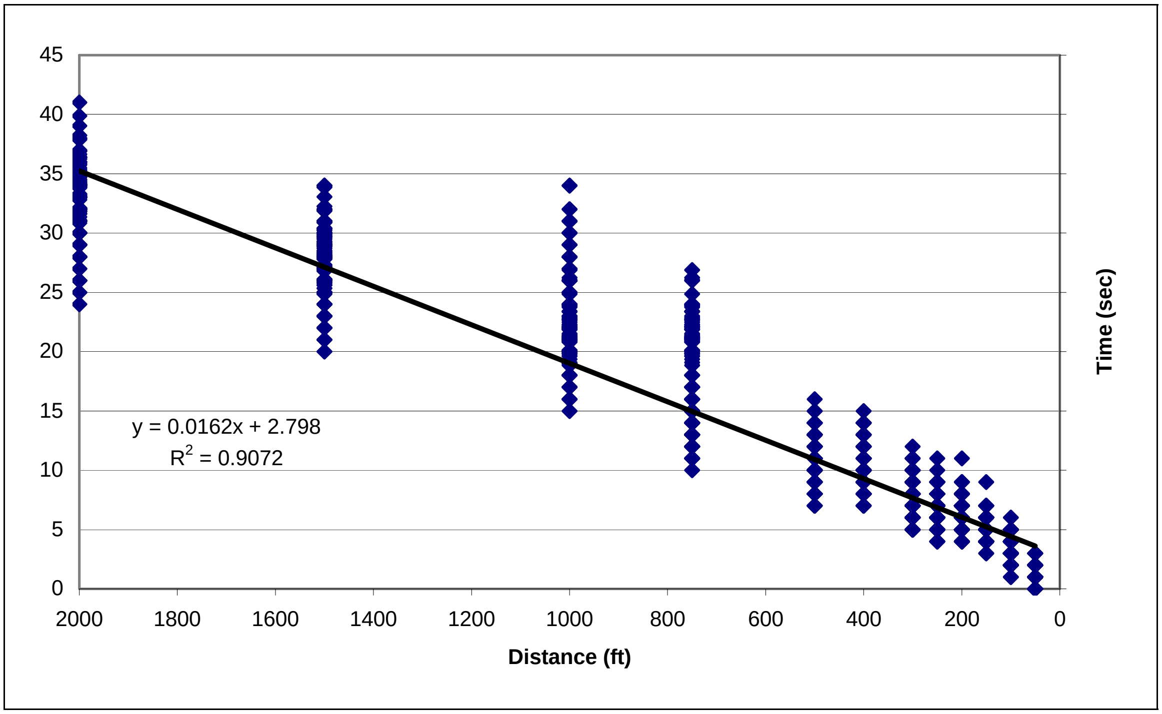

Figure 7.8. Time of Free-Flow Passenger Cars in Dry Conditions Traveling 55-59 mph at 2000 ft from a Stop at Tangent Sites

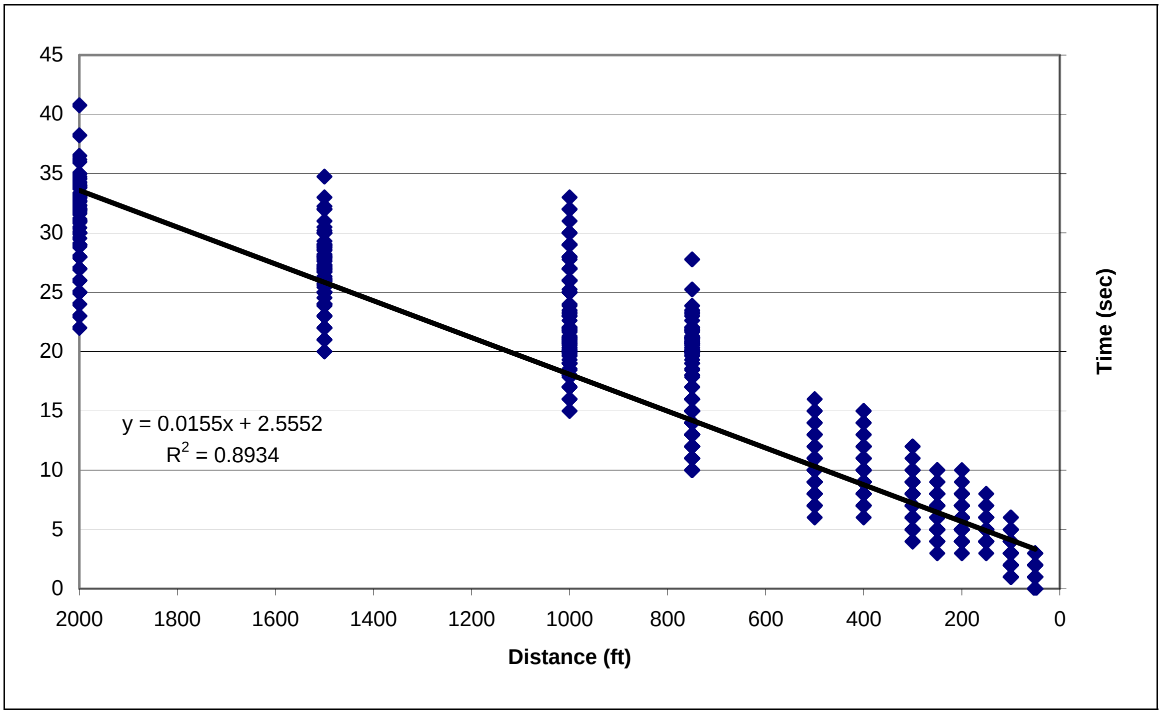

Figure 7.9. Time of Free-Flow Passenger Cars in Dry Conditions Traveling 60-64 mph at 2000 ft from a Stop at Tangent Sites

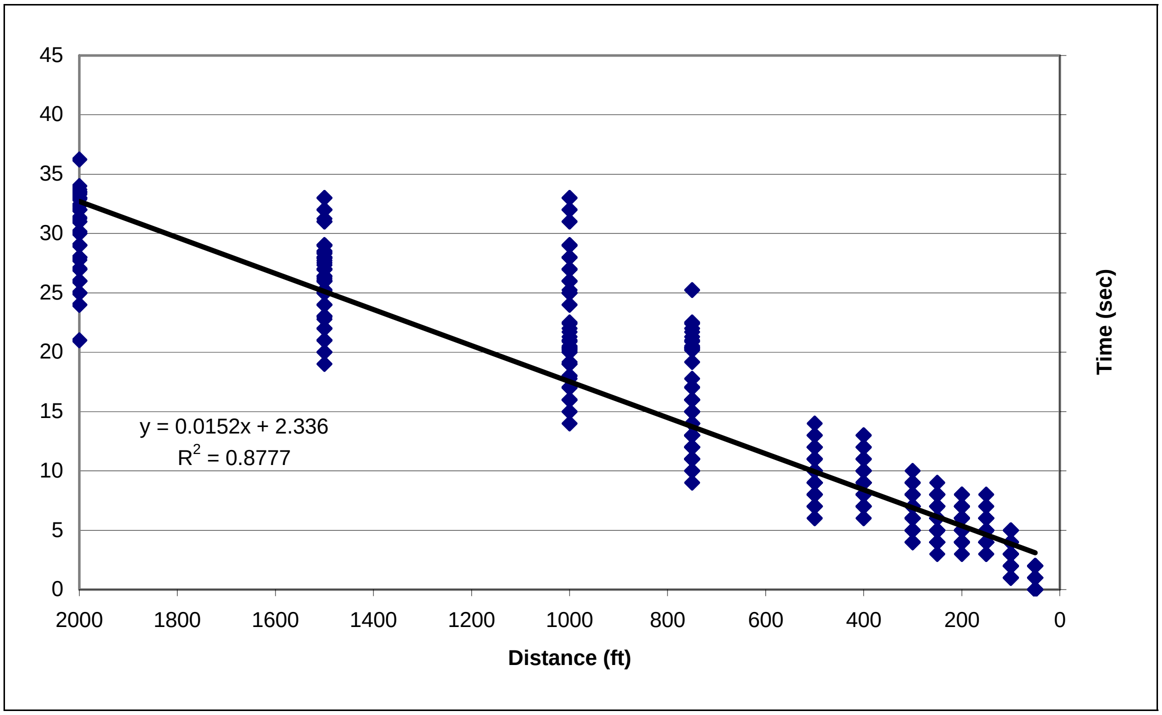

Figure 7.10. Time of Free-flow Passenger Cars in Dry Conditions Traveling 65-69 mph at 2000 ft from a Stop at Tangent Sites

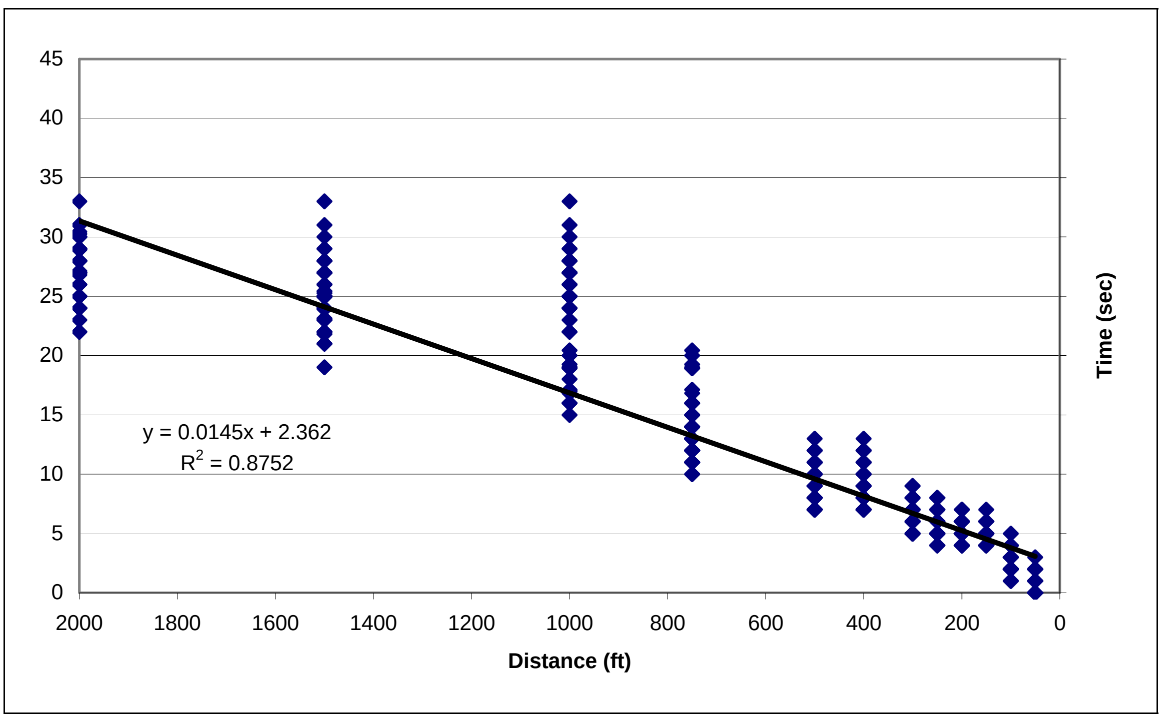

Figure 7.11. Time of Free-flow Passenger Cars in Dry Conditions Traveling 70 mph and Greater at 2000 ft from a Stop at Tangent Sites

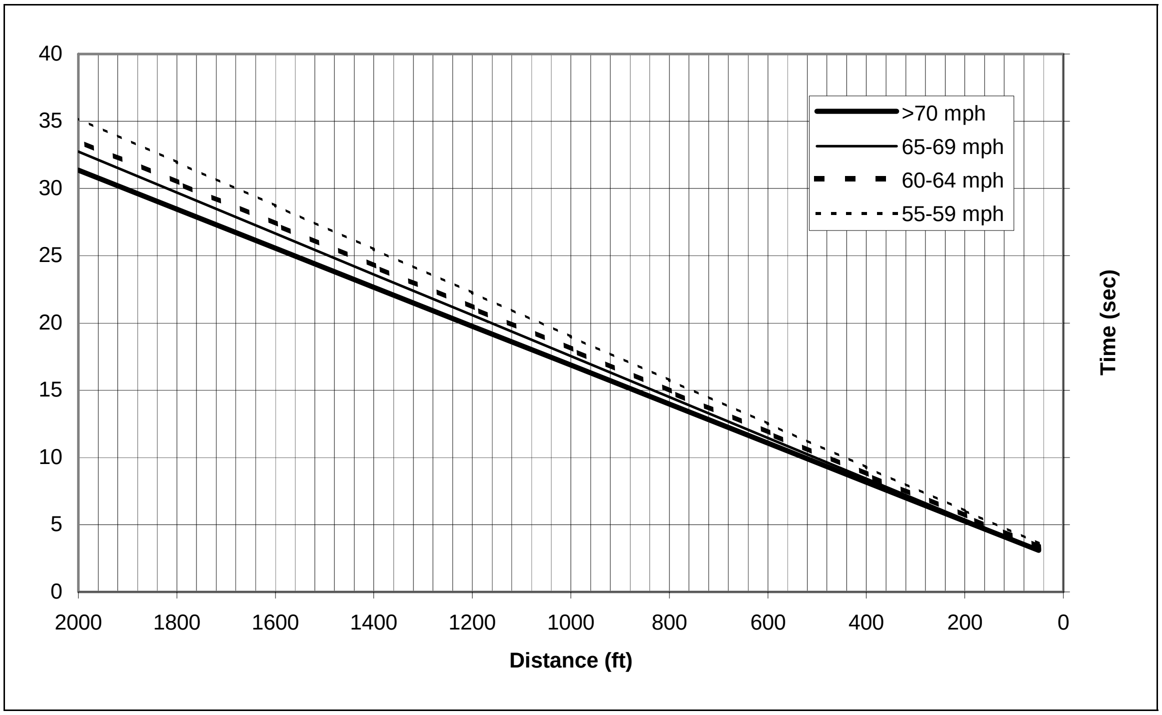

Figure 7.12. Time of Four Entry Speed Categories of Free-flow Passenger Cars in Dry Conditions Approaching a Stop at Tangent Sites

The iterative procedure developed in this research results in a minimum recommended radius that meets all of the six requirements listed earlier in this chapter. There are several input variables this procedure uses to calculate an appropriate design. These variables are:

- central angle of the curve (deflection angle of tangents), Δ,
- profile grade of the crossroad, P_{CR},
- profile grade at the point of tangency on the approach roadway, P_{PT},
- design speed of the approach facility, V,
- width of the approach lane (desirably 12 ft in Nebraska), w,
- number of lanes rotated through transition (1 for a two-lane two-way highway), n,
- normal crown cross slope (desirably 2% in Nebraska),
- maximum superelevation at the PC on the approach roadway (6% used in procedure), and
- maximum superelevation at the PT on the approach roadway (4% used in procedure).

Once a minimum radius value is calculated that reasonably corresponds with the speed profile associated with vehicles approaching the stop from a speed which is near the design speed of the curve throughout its length, the following characteristics are investigated:

- the rate of change in lateral acceleration from the PC to the PT,
- the braking distance from the PT to the stop, and
- the rate of deceleration from the PT to the stop.

These checks determine if a chosen radius is appropriate for the given conditions. Table 7.3 displays the values that determine an appropriate design in terms of rate of change in lateral acceleration and rate of deceleration. An appropriate braking distance occurs when the length needed to transition superelevation from the cross slope of the approach roadway at the PT to match the profile grade of the crossroad is greater than the distance required for the sum of perception-reaction distance and braking distance, assuming a 1-second perception-reaction time. All three features must meet these requirements for a chosen radius to be considered appropriate.

Table 7.3. Appropriate Values for Rate of Change in Lateral Acceleration and Rate of Deceleration

	Rate of Change in Lateral Acceleration from the PC to the PT, ft/s^3	Rate of Deceleration from the PT to the Stop, mph/sec
Desirable	1-3	
Acceptable	4	≤ 7.6
Inappropriate	> 4	

Source: (*4*)

Factors Measuring Design Appropriateness

Rate of Change in Lateral Acceleration

One factor to consider when driving on a horizontal curve is the driver's comfort level. The relationship that best quantifies driver comfort is displayed in Equation 7.1.

$$a_f = f_C g \qquad \textbf{(7.1)}$$

where,

a_f = lateral acceleration, ft/sec^2,
f_C = side friction factor, dimensionless, and
g = gravitational constant, ft/sec^2.

The side friction factor represents the tires' resistance to lateral acceleration that acts on the vehicle (*4*). Driver comfort and lateral acceleration become especially important when high speeds are combined with sharp curves. The solution to this problem may result in the use of a spiral curve. The advantage of a spiral curve is that it provides a natural easy-to-follow path for drivers and smooth transitions to and from the curve. The transition section of a spiral path by the vehicles should correspond to the rate of change in lateral acceleration. This rate of change in lateral acceleration is similar to the one drivers experience on a circular curve approaching a stop as they transition from higher speeds to lower speeds. The 2001 Green Book has assigned desirable and acceptable values for the rate of change in lateral acceleration reported in Table 7.3. These rates help determine if a curve is too sharp to ensure driver comfort.

Stopping Distance

The next factor that is used to evaluate curve geometry is stopping distance, which is a combination of perception-reaction distance and braking distance. Stopping distance is the sum of the distance traversed during the perception-reaction time and the distance to brake the vehicle to a stop (*4*). Perception-reaction time is defined by the 2001 Green Book as "the interval from the instant that the driver recognizes the existence of an obstacle on the roadway ahead that necessitates braking to the instant that the driver actually applies the brakes (*4*)." The four different responses associated with perception-reaction time are perception, intellection, emotion, and volition. Perception refers to the driver's first detection of a possible obstacle, and intellection occurs when the driver recognizes the obstacle. Emotion determines the decision of how to appropriately respond to an obstacle and volition refers to the driver's actual response to the obstacle.

Determining an appropriate perception-reaction time for this study is important in order to calculate an adequate amount of perception-reaction braking distance between the PT and the vehicle's stopping point. The 2001 Green Book considers a perception-reaction time of 2.5 seconds as an adequate amount of time to react in unexpected situations. Research has shown that 2.5 seconds exceeds the 90th-percentile stopping sight distance perception-brake reaction time for all drivers (*20*). The procedure developed by this research assumes that drivers will be aware of the need to stop by the time they reach the PT and will already begin to brake within the limits of the curve. Because drivers are assumed to be aware of the intersection before they reach the PT, the braking maneuver will be expected. An expected obstacle is one that has been detected and recognized by the driver. For this reason, the driver should only need time to decide and react to an obstacle. Research conducted by Fambro et al. (*20*) studied the relationship between an expected stop and perception-reaction time (PRT). Table 7.4 reports the summary of their findings.

Table 7.4. Summary of Perception-Response Time to an Expected Object

Study #	Age	Gender	No. of Test Subjects	Total No. Repetitions	Mean PRT (sec)	Standard Deviation (sec)
Study 2	Older	Female	7	134	0.66	0.216
		Male	7	129	0.65	0.228
	Younger	Female	6	117	0.57	0.167
		Male	6	113	0.48	0.088
Study 3	Older	Female	5	90	0.67	0.252
		Male	3	52	0.65	0.345
	Younger	Female	2	40	0.49	0.168
		Male	1	20	0.55	0.078

Source: (*20*)

In order to make these data comparable to the AASHTO PRT, the 95th-percentile PRT needed to be calculated. Table 7.5 displays the 95th-percentile PRT assuming a normal distribution. These data suggest that 95 percent of drivers should be able to perceive and react to an expected obstacle within one second. This also reinforces the idea that less time is needed for the PRT because the driver has already detected and perceived the stop condition ahead and only needs time to decide and react to it. Fambro et al. (*20*) concluded that the AASHTO PRT of 2.5 seconds should be used in design but also noted, "shorter perception-brake reaction times may be appropriate for traffic signal design where change intervals are expected (*20*)." For these reasons, a perception-reaction time of 1.0 second was chosen to calculate perception-reaction braking distance.

Table 7.5. 95th-Percentile Perception Reaction Time to an Expected Object

Study #	Age	Gender	Mean PRT (sec)	Standard Deviation (sec)	95th %ile PRT (Z=1.645) (sec)
Study 2	Older	Female	0.66	0.216	1.015
		Male	0.65	0.228	1.025
	Younger	Female	0.57	0.167	0.845
		Male	0.48	0.088	0.625
Study 3	Older	Female	0.67	0.252	1.085
		Male	0.65	0.345	1.218
	Younger	Female	0.49	0.168	0.766
		Male	0.55	0.078	0.678

A consistent location for the stop location of the vehicle approaching the intersection was required for the procedure. National Cooperative Highway Research Program (NCHRP) Report 383 (*21*) determined 6.6 ft to be the 85th-percentile stopping position from the edge line of the crossroad. The report also suggests that a more generous design is desirable by stating, "it is recommended that the distance from the edge of the major-road traveled way to the front of the stopped vehicle should be at least 6.6 ft and, where feasible, 10 ft." For this research, the vehicle's stopping point is assumed to be 10 ft back from the lane edge of the crossroad based upon the recommendation of NCHRP Report 383 (*21*).

Deceleration Rate

Another factor that is used to determine if a design produced by the procedure is appropriate is the rate of deceleration between the PT and vehicle's stopping point. The 2001 Green Book suggests that a deceleration rate of 7.6 mph/sec (11.2 ft/sec^2) be used as a comfortable deceleration for most drivers in unexpected situations. Recent research by Fambro et al. (*20*) studied the maximum deceleration to an expected object under different driving conditions. The results of the studies are displayed in Table 7.6. The research found the average of the mean maximum deceleration on curves in dry conditions to be 15.1 mph/sec. They also found the average of the mean maximum deceleration on curves in wet conditions to be 13.7 mph/sec.

Table 7.6. Summary of Findings by Fambro et al.

Study	ABS*	Pavement	Geometry	Mean Max Decel (mph/sec)	Average of the Means Max Decel (mph/sec)
2	no	Dry	Curve	14.93	
2	yes	Dry	Curve	16.03	15.1
3	no	Dry	Curve	14.49	
2	no	Wet	Curve	13.39	
2	yes	Wet	Curve	14.93	13.7
3	no	Wet	Curve	12.73	

Source: (*20*) *-Anti-lock Braking System

Speeds of vehicles were collected on roadway tangents to better understand the relationship between deceleration and driver comfort in preferred driving conditions. Speed data for this research was measured at thirteen different points on the tangent approaching a stop. Deceleration rates were then calculated between each point and the 85th-percentile and 15th-percentile deceleration rates between detectors was determined. Once these values were calculated, regression analysis was used to determine the best-fit line representing the data points. The results of this analysis along with values from previous research are displayed in Figure 7.13.

The results indicate that the observed data reinforces the 2001 Green Book value of 7.6 mph/sec as a comfortable deceleration for most drivers. For this reason, 7.6 mph/sec was used as the desirable deceleration rate.

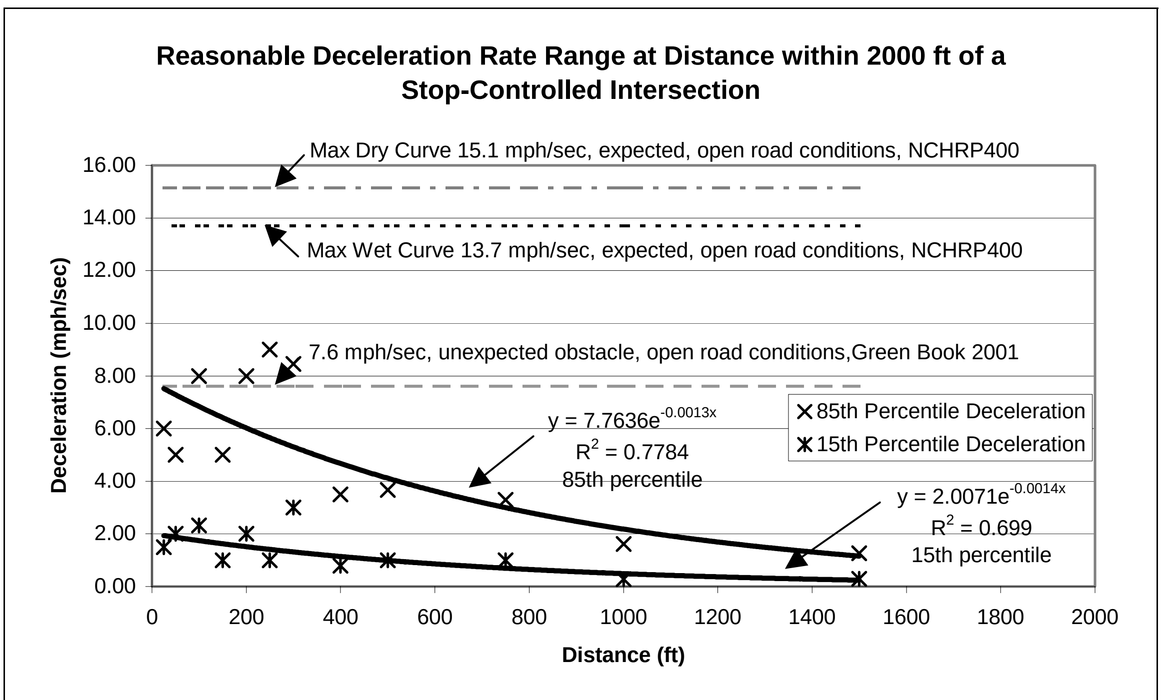

Figure 7.13. 85th- and 15th-Percentile Deceleration Regression Results

Factors of Importance

Design Speed

AASHTO defines design speed as “a selected speed used to determine the various geometric design features of the roadway (*4*).” NCHRP Report 439 defines “curve design speed” as the expected 95th-percentile speed of freely flowing passenger cars on a curve (10). Design speeds are selected based on many different factors such as adjacent land use, topography, and functional classification of the highway. This research will include four different design speed situations. The design speeds chosen to evaluate in this procedure are 55 mph, 60 mph, 65 mph, and 70 mph, which are representative of the design speeds used in Nebraska for rural two-lane, two-way highways.

Side Friction

As mentioned previously, side friction is the force developed when centripetal acceleration is unbalanced by superelevation. Designers use the concept of maximum side friction factor to determine the speed on a curve at which discomfort due to the lateral acceleration becomes evident to drivers. This is the point where drivers react instinctively to avoid higher speeds (*4*).

The focus of this research is on horizontal curves that approach a stop-controlled intersection on two-lane two-way rural highways. Intuitively, this research would use the maximum side friction values for intersection curves reported in the 2001 Green Book. Table 7.7 duplicates Exhibit 3-43 of the 2001 Green Book.

Table 7.7. Minimum Radii for Intersection Curves

Design (turning) speed V (mph)	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**
Side Friction Factor, f	0.38	0.32	0.27	0.23	0.20	0.18	0.16	0.15
Assumed minimum superelevation, e/100	0.00	0.00	0.02	0.04	0.06	0.08	0.09	0.10
Total e/100 + f	0.38	0.32	0.29	0.27	0.26	0.26	0.25	0.25
Calculated minimum radius, R (ft)	18	47	92	154	231	314	426	540
Suggested minimum radius curve for design (ft)	25	50	90	150	230	310	430	540
Average running speed (mph)	10	14	18	22	26	30	34	36

Note: For design speeds greater than 45 mph, use values for open highway conditions

Source: (*4*)

The superelevation values for design speeds greater than 30 mph are based on superelevations which exceed the desired superelevation (0.06 ft/ft) of rural horizontal curves in Nebraska. For this reason, this research chose to use the maximum side friction values for low-speed urban streets reported in the 2001 Green Book. Table 7.8 displays these values, summarized from Exhibit 3-41 in the 2001 Green Book. These values correspond to low speed conditions (speeds < 50mph), which are believed to be most applicable to friction factors of vehicles approaching a stop condition.

Table 7.8. Maximum Side Friction Values for Low-Speed Conditions

Design Speed (km/h)	Design Speed (mph)	Max e/100	Max f	Total (e/100 + f)	Min R (m)	Min R (ft)
20	12.4	0.06	0.350	0.410	10	32.8
30	18.6	0.06	0.312	0.372	20	65.6
40	24.9	0.06	0.252	0.312	40	131.2
50	31.1	0.06	0.214	0.274	70	229.7
60	37.3	0.06	0.186	0.246	115	377.3
70	43.5	0.06	0.163	0.223	175	574.1

Source: (*4*)

Maximum side friction values for high speed conditions (speeds ≥ 50 mph) are taken from the 2001 Green Book's design of rural highways. These values are displayed in Table 7.9 and summarized from Exhibit 3-14 in the 2001 Green Book.

Table 7.9. Maximum Side Friction Values for High-Speed Conditions

Design Speed (km/h)	Design Speed (mph)	Max e/100	Max f	Total (e/100 + f)	Min R (m)	Min R (ft)
80	49.7	0.06	0.140	0.200	251.8	826.1
90	55.9	0.06	0.130	0.190	335.5	1100.7
100	62.1	0.06	0.120	0.180	437.2	1434.4
110	68.4	0.06	0.110	0.170	560.2	1837.9
120	74.6	0.06	0.090	0.150	755.5	2478.7
130	80.8	0.06	0.080	0.140	950.0	3116.8

Source: (*4*)

Input Variables

Several input variables need to be chosen for the procedure. Some of these variables include crossroad profile grade, profile grade at the PT, and superelevation. These variables are controlled by state and/or national standards. Controlled values serve as a boundary for what design combinations of input variables are considered feasible.

Grade

The State of Nebraska Minimum Design Standards lists 6.5 as the maximum percent grade for the lowest volume rural highway type (*22*). Therefore, maximum values for the crossroad profile grade and the profile grade at the PT are 6.5% and –6.5%.

Superelevation

Maximum rates of superelevation are controlled by the following factors:

- climate conditions – frequency and amount of snow and ice,
- terrain conditions – flat through mountainous,
- type of area – rural or urban, and
- frequency of very slow-moving vehicles that would be subject to uncertain operation.

The NDOR Roadway Design Manual lists 6% as the desirable superelevation for rural roads (*1*). The Green Book states that "when traveling slowly around a curve with high superelevation, negative lateral forces develop and the vehicle is held in the proper path only when the driver steers up the slope or against the direction of the horizontal curve. Steering in this direction seems unnatural to the driver and may explain the difficulty of driving on roads where the superelevation is in excess of that needed for travel at normal speeds." This research chose to use a 4% superelevation rate from near the midpoint of the horizontal curve through the PT because vehicles will be reducing their speed as they approach a stop condition at an intersection. The 6% superelevation should be transitioned to full superelevation at the PC end of the curve then immediately rotated down to the 4% maximum rate through the majority of the curve length until the superelevation must transition again at the PT end of the curve.

Other variables will also be used in this procedure, but their values are not controlled by state or national standards and therefore will be discussed in later sections.

Chapter 8
DESIGN PROCEDURE

Initial Input Values

Before the procedure can begin, the general intersection situation characteristics must be known.

1. **The design speed or predicted 95th-percentile speed of the overall roadway facility must be determined.** If the roadway intersection segment to be designed is an existing roadway, estimates of this value can be made by using 95th-percentile speed prediction equations developed in NDOR Research Project SPR-PL-1(36) P519, *Relationship Between Design, Operating, and Posted Speed Under High-Posted Speed Conditions* (*23*). The design speed or predicted 95th-percentile speed will dictate which of the four speed category profiles will be used to best fit the horizontal curve to be designed with decelerating vehicle speeds.
2. **The desired central angle (deflection between tangents) must be known.** A small central angle will create shorter curve lengths, while large central angles create longer curve lengths due to the geometric properties of circular curvature.
3. **The profile grade of the crossroad at the intersection must be estimated.** This value determines the distance of approach roadway needed to transition from a normal crown cross section to match the profile grade at the intersection. Since the exact location of the intersection is determined by the procedure, the crossroad profile grade must be estimated initially.
4. **The profile grade at the point of tangency of the curve must be estimated.** This value will contribute to length of the braking distance required from the PT of the curve to the stop. The profile grade at the PT must be estimated at the beginning of the procedure.

Procedure

The procedure is a step-by-step iterative process that determines a minimum radius of curvature that also minimizes the amount of right-of-way required to connect the skewed intersection alignments at a 90-degree angle while fulfilling current design and safety standards.

Step 1

A. **Select a "design" speed (V) for the horizontal curve.** Desirably, the speed chosen should approximate the 95th-percentile speed of free-flow passenger cars on the roadway at the location where the curve will be ultimately be placed.

B. **Select a superelevation rate that will serve as the maximum superelevation (e_{max}) at the PC and PT end of the curve.** This value should be less than the superelevation rate at the PC end of the curve because the speed of vehicles traversing the curve will be less at the PT than the PC due to the fact that vehicles in this situation are approaching a stop-controlled intersection. The superelevation maximum at the PC should be the desirable rate prescribed by the NDOR standards which is 0.06 ft/ft. Once the maximum rate is attained, the cross slope should immediately be rotated to the lower superelevation maximum of 0.04 ft/ft.

C. **Determine the corresponding maximum side friction factor (f_{max}) for the speed chosen in *Part A.* of *Step 1.*** Table 8.1 displays the f_{max} values used in this procedure along with equations that have been developed to interpolate values between the incremental f_{max} values given in the 2001 Green Book. Figure 8.1 graphically displays the relationship between curve speed and maximum side friction and the corresponding segmental equations.

Table 8.1. Maximum Side Friction Factors Utilized by the Procedure

Speed (kph)	Speed (mph)	f_{max}	Linear equation $y = f_{max}$ x = Speed (mph)	Source
20	12.43	0.350		
			$y = -0.0061x + 0.426$	
30	18.64	0.312		
			$y = -0.0097x + 0.492$	
40	24.85	0.252		Low Speed Urban Street
			$y = -0.0061x + 0.404$	Friction Factors, p. 197, 2001 GB
50	31.07	0.214		
			$y = -0.0045x + 0.354$	
60	37.28	0.186		
			$y = -0.0037x + 0.324$	
70	43.50	0.163		
				Interpolated Values
80	49.72	0.140		
90	55.94	0.130		
			$y = -0.0016x + 0.220$	High Speed Rural Highway
100	62.15	0.120		Friction Factors, p. 145, 2001 GB
110	68.37	0.110		
			$y = -0.0032x + 0.330$	
120	74.58	0.090		

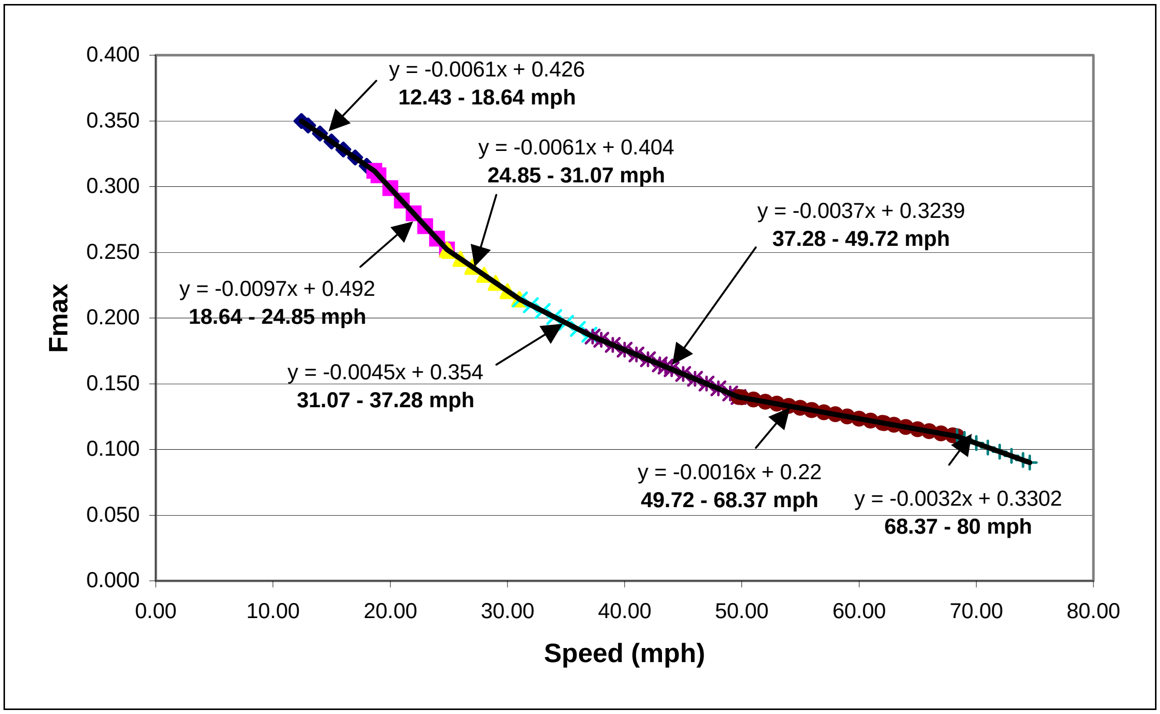

Figure 8.1. Graphical Display of Maximum Side Friction Factors

Step 2

A. **Calculate the minimum radius of the curve utilizing speed, superelevation, and the f_{max} from *Step 1*.** Equation 8.1 shows the equation used to calculate the minimum radius.

$$R_{min} = V^2/14.90(e + f) \qquad \textbf{(8.1)}$$

where,

R_{min} = minimum traveled path radius allowable to provide driver comfort, ft,
V = curve design speed, mph,
e = maximum superelevation rate, ft/ft, and
f = side friction factor (from Table 8.1).

B. **The tangent length of the curve is the next geometric feature that must be calculated.** Equation 8.2 displays the tangent length formula.

$$T = R_{min} \tan\left(\frac{\Delta}{2}\right) \qquad \textbf{(8.2)}$$

where,

T = tangent length, ft,
R_{min} = minimum traveled path radius allowable to provide driver comfort, ft, and
Δ = central (deflection) angle, degrees.

C. **The arc length of the curve must be calculated next.** Equation 8.3 displays the curve length formula.

$$L_c = \frac{\pi R_{min} \Delta}{180} \quad \textbf{(8.3)}$$

where,

L_c = horizontal curve length, ft,
π = Pi, dimensionless,
R_{min} = minimum traveled path radius allowable to provide driver comfort, ft, and
Δ = central (deflection) angle, degrees.

D. **Length of superelevation runoff at the PT is the next value to calculate.** The superelevation runoff is the length of roadway needed to accomplish a change in outside-lane cross slope from zero (flat) to full superelevation, or vice versa (*4*). Equation 8.4 displays the formula for length of superelevation runoff used in the 2001 Green Book (*4*).

$$L_r = \frac{w(n)(e_{max})(b_w)}{\text{max relative gradient}} \quad \textbf{(8.4)}$$

where,

L_r = length of superelevation runoff, ft,
w = width of the approach lane, ft,
n = number of lanes rotated,
e_{max} = maximum superelevation, percent,
b_w = adjustment factor for the number of lanes rotated, and
max relative gradient = maximum relative gradient, dimensionless.

Width of the approach lane is typically 12 ft. The number of lanes rotated may vary but for this research only two-lane rural highways were considered (n = 1). Maximum superelevation is the same maximum superelevation rate chosen in *Step 1*. The adjustment factor for the number of lanes rotated for this research is 1 which corresponds to one lane of rotation according to Exhibit 3-28 of the 2001 Green Book. Values from that exhibit are duplicated in Table 8.2

Table 8.2. Adjustment Factors for the Number of Lanes Rotated

Number of Lanes Rotated n	**Adjustment Factor b_w**
1	1.00
1.5	0.83
2	0.75
2.5	0.70
3	0.67
3.5	0.64

Source: (*4*)

The choice for maximum relative gradient is based upon the "design" speed chosen in *Step 1* and represents the maximum acceptable difference between the longitudinal grades of the axis of rotation and the edge of the lane (*4*). Table 8.3 displays the values of maximum relative gradient for incremental design speeds which are duplicated from Exhibit 3-27 in the 2001 Green Book.

Table 8.3. Maximum Relative Gradients

Design Speed (mph)	**Maximum Relative Gradient (%)**
15	0.78
20	0.74
25	0.70
30	0.66
35	0.62
40	0.58
45	0.54
50	0.50
55	0.47
60	0.45
65	0.43
70	0.40
75	0.38
80	0.35

Source: (*4*)

E. **The length of tangent runout at the PT must be calculated next.** The tangent runout is the length of roadway needed to accomplish a change in outside-lane cross slope from the normal cross slope rate to zero (flat), or vice versa (*4*). This configuration of roadway cross slope is commonly referred to as adverse crown removed (ACR) (*4*). Equation 8.5 displays the formula for tangent runout used in the 2001 Green Book.

$$\tan\ \text{runout} = \left(\frac{e_{\text{normal crown}}}{e_{\max}} \right) * L_r \qquad \textbf{(8.5)}$$

where,

tan runout = length of the tangent runout, ft,
e_{max} = maximum superelevation, percent,
$e_{normal\ crown}$ = superelevation of the normal crown, percent, and
L_r = length of superelevation runoff, ft.

The value for maximum superelevation is the same as in *Step 1*. The term normal crown refers to a roadway which is peaked in the middle with equal cross slopes on either side. The typical slope value of the normal crown in Nebraska is 2%. The quantity calculated previously should be multiplied by 2 to rotate the roadway from normal crown up or down to the profile grade of the crossroad pavement edge.

F. **The transition length from adverse crown removed (ACR) at the PT to the profile grade of the crossroad is the next value to be calculated.** Equation 8.6 displays the formula for transition from ACR to the profile grade of the crossroad.

$$\text{transition} = \frac{w\left(PG_{cr}\right)}{\text{max relative gradient}} \quad \textbf{(8.6)}$$

where,

w	=	width of the roadway, ft,
PG_{cr}	=	profile grade of the crossroad, percent, and
max relative gradient	=	maximum relative gradient, dimensionless.

G. **The distance from the point of tangency to the crossroad pavement edge is then determined.** The distance to transition from normal crown up or down to the crossroad pavement edge and transition length to the portion of superelevation runoff that occurs prior to the curve must be added together. The portion of runoff located prior to the curve depends on the design speed or the 95th-percentile speed of the facility along with the number of lanes rotated. Table 8.4 displays the values for the portion of runoff located prior to the curve according to Exhibit 3-30 of the 2001 Green Book. Equation 8.7 displays the formula for calculating the distance from the point of tangency to the crossroad pavement edge.

Table 8.4. Portion of the Runoff Located Prior to the Curve

Design speed (mph)	**Portion of runoff located prior to the curve — Number of lanes rotated (n): 1.0**	**1.5**	**2.0-2.5**	**3.0-3.5**
15-45	0.80	0.85	0.90	0.90
46-49*	0.75	0.80	0.85	0.88
50-80	0.70	0.75	0.80	0.85

*-interpolated values

Source: (*4*)

Dist = (tan runout)(2) + (transition) + (portion of runoff located prior to curve)(L_r) **(8.7)**

H. **The distance from the point of tangency to the stop should be calculated next.** The stop condition is considered to be 10 feet back from the crossroad edge of pavement. This means the distance can be determined by subtracting 10 feet from the distance from the point of tangency to the crossroad pavement edge determined in *Step 2, Part G*.

I. **Once the distance from the PT to the stop has been determined, the distance from the PC to the stop needs to be calculated.** This can be done by adding the curve length, L_c to the distance from the PT to stop.

Step 3

The next step is to utilize the speed profile equations given in Table 7.1 to calculate the expected speed at the PC determined by the initial choice of horizontal curve.

A. **Once the expected speed at the PC is calculated, it can then be compared to the "maximum acceptable speed" at the point of curvature.** The maximum acceptable speed at the point of curvature is determined by the minimum radius equation, Equation 8.1. As discussed in *Step 1, Part B*, the recommended value for e_{max} at the PC is 0.06 ft/ft. This superelevation maximum will be larger than the one at the PT (0.04 ft/ft) because the speed of a vehicle entering at the PC will be greater than at the PT. Intuitively, the incorporation of two maximum superelevations on the horizontal curve makes sense because at one end vehicles are entering at higher speeds. At the other end of the curve, the vehicles will be slowing down considerably to negotiate the stop situation ahead.

The radius for this horizontal curve has been calculated in *Step 1*. This leaves speed and side friction factor as the only unknowns in the minimum radius equation. To find the maximum acceptable speed at the PC with the superelevation rate for the PC of 0.06 ft/ft, another set of equations needs to be introduced to solve for the two unknowns. The equation used to complete this system of linear equations can be determined through the side friction factors listed in *Step 1*. When linear relationships are regressed between speed and side friction, the system of equations can be completed and solved simultaneously. Table 8.5 displays these linear relationships and the ranges of speeds for which they apply.

Table 8.5. Linear Relationships Between Side Friction Factor and Speed

Speed Range (kmph)	Speed Range (mph)	Linear Equation $y = f_{max}$ x = Speed (mph)
20-30	12.43-18.64	y = -0.0061x + 0.426
30-40	18.64-24.85	y = -0.0097x + 0.492
40-50	24.85-31.07	y = -0.0061x + 0.404
50-60	31.07-37.28	y = -0.0045x + 0.354
60-80	37.28-49.72	y = -0.0037x + 0.3239
80-110	49.72-68.37	y = -0.0016x + 0.22
110-120	68.37-74.58	y = -0.0032x + 0.3302

B. **Next, the equations in Table 8.5 and that for the minimum radius can be rearranged solving for speed.** Equations 8.8 and 8.9 display the new formulas.

$$V_{\text{max acc PC}} = \frac{f_{max} - b}{a} \tag{8.8}$$

where,

$V_{\text{max acc PC}}$ = maximum acceptable speed at the PC, mph,
f_{max} = maximum side friction factor, dimensionless,
a = slope of the linear equation from Table 8.5, dimensionless, and
b = intercept of the linear equation from Table 8.5, dimensionless.

$$V_{\text{max acc PC}} = \sqrt{R_{min}\left(15\left(e_{\text{max PC}} + f_{max}\right)\right)} \tag{8.9}$$

where,

$V_{\text{max acc PC}}$ = maximum acceptable speed at the PC, mph,
R_{min} = minimum traveled path radius allowable to provide driver comfort, ft,
$e_{\text{max PC}}$ = maximum superelevation rate at the PC, percent, and
f_{max} = maximum side friction factor, dimensionless.

Once these equations are solved simultaneously, the maximum acceptable speed at the PC can be determined.

C. **Next, the expected speed at the PC and the maximum acceptable speed at the PC are compared.** If the maximum acceptable speed is less than the expected speed, the design is inadequate and the process must start over at *Step 1*. Starting over from *Step 1*, the “design” speed of the curve must be increased and a new radius calculated.

D. **This process must be repeated until the maximum acceptable speed at the PC is greater that the expected speed at the PC.** When this occurs, the facility will be able to conservatively accommodate the vehicle's expected speed throughout the entire curve. To minimize the horizontal curve radius along with the amount of right-of- way needed for construction, the difference between maximum acceptable speed at the PC and expected speed at the PC should be as small as possible.

Step 4
The next step in the procedure is to calculate the expected speed at the point of tangency. This value can be determined with the distance from the stop calculated in *Step 2, Part H* along with the appropriate speed profile equation in Table 7.1.

A. **The next item to calculate is the stopping distance required for perception-reaction and braking.** Vehicles traveling on horizontal curves do not have full friction available for braking, but instead have a reduced amount because of the side friction already demanded of the contact patch of the tire and the pavement surface in cornering (*24*). The first item that needs to be calculated is the available friction for stopping when the vehicle is still negotiating the curve at the PT. Equation 8.10 displays the formula for available friction for stopping. Figure 8.2 shows the side friction and braking friction components of the total friction available between the contact patch of the tires and the pavement surface.

$$f_B' = (f_B^2 - f_C^2)^{0.5} \quad \textbf{(8.10)}$$

where,

f_B' = available friction for stopping, dimensionless,
f_B = braking friction factor, dimensionless, and
f_C = maximum side friction factor, dimensionless.

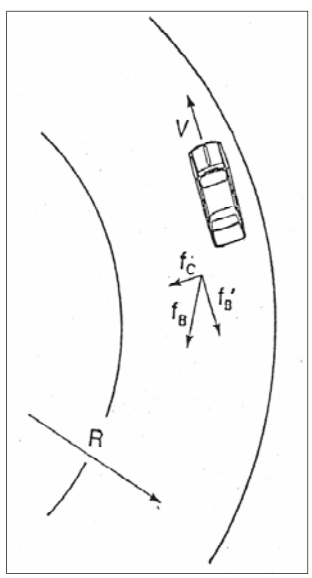

Figure 8.2 Components of Friction When Braking on a Horizontal Curve (*24*)

The maximum side friction factor for Equation 8.10 is the iterated side friction factor calculated in *Step 3, Part B* when the maximum acceptable speed is greater than the expected speed. The braking friction factor for this equation comes from the 1990 edition of the Green Book (*25*). The reason this friction factor is used is because the later editions of the Green Book do not use side friction to calculate braking distance, but instead use a maximum deceleration rate. Table 8.6 displays the values from the 1990 Green Book for the braking friction factor that are based on the expected speed at the PT.

Table 8.6. Braking Friction Factors

Expected Speed at the PT (mph)	Braking Friction Factor
20	0.40
25	0.38
30	0.35
35	0.34
40	0.32
45	0.31
50	0.30
55	0.30
60	0.29
65	0.29
70	0.28

Source: (*25*)

The next item to calculate is the portion of braking friction available. This can be determined by dividing the braking friction factor, f_B' by the available friction for stopping, f_B .

B. **Next, the braking distance needs to be calculated.** Equation 8.11 displays the formula for braking distance on a grade.

$$d = \frac{V^2}{30\left(\left(\frac{a}{32.2}\right) \pm G\right)\left(\frac{f_B'}{f_B}\right)} \qquad \textbf{(8.11)}$$

where,

d	=	braking distance, ft,
V	=	Speed at the PT, mph,
a	=	deceleration rate, ft/sec^2,
G	=	grade at the PT, percent,
f_B'	=	available friction for stopping, dimensionless, and
f_B	=	braking friction factor, dimensionless.

The braking distance equation is modified from the equation given in the 2001 Green Book. The equation in the 2001 Green Book assigns all friction to the braking maneuver. Equation 8.11 takes into account that a portion of side friction is being used to offset lateral acceleration at the PT. The portion of side friction that is available for braking is incorporated into the equation to reflect the actual braking distance required when the vehicle is at the PT location. The 2001 Green Book suggests the use of 11.2 ft/s^2 as the deceleration rate ("a" in Equation 8.11).

C. **Lastly, the distance traveled during perception reaction needs to be calculated.** Equation 8.12 displays the formula for perception-reaction distance.

$$\text{PR distance} = 1.47\left(V_{exp,PT}\right)(t) \qquad \textbf{(8.12)}$$

where,

PR distance	=	perception-reaction distance, ft,
$V_{exp,PT}$	=	expected speed at the PT, mph, and
t	=	perception-reaction time, sec.

The expected speed at the PT can be determined using the appropriate speed profile equation from Table 7.2.

D. Now, the values obtained from Equations 8.11 and 8.12 can be added together to obtain the perception-reaction/braking distance.

E. **Next, the perception-reaction/braking distance must be compared with the distance from the PT to the stop calculated in *Step 2, Part H*.** The perception-reaction/ braking distance must be smaller than the distance needed for transition. If it is not, the driver may have an insufficient length of roadway in which to stop. If it is, the driver should have a sufficient length of roadway in which to stop as well as a sufficient amount of roadway for superelevation transition. Several design characteristic values may need to be changed in the event that the perception-reaction/braking distance is less than the distance needed for transition. Design characteristic values that affect these distances are: the design speed chosen in *Step 1*, the profile grade at the PT, and the profile grade of the crossroad. To move onto the next step, one or a combination of these values needs to change to produce a design in which the perception-reaction/braking distance is less than the distance needed for transition.

Step 5
At this point the design of the facility is done, but the design itself needs to be evaluated by the criteria discussed in Chapter 7, and summarized in Table 7.1.

A. **To begin the evaluation process, the lateral acceleration of a vehicle on the curve at the PC needs to be calculated.** Equation 8.13 displays the formula for the vehicle's lateral acceleration on a curve expressed as a point mass (*26*).

$$\text{Accel on Curve} = \frac{V_{exp,PC}{}^{2}\left(\frac{5280^2}{3600^2}\right)}{R_{min}} \qquad \textbf{(8.13)}$$

where,

Accel on Curve	=	vehicle's lateral acceleration on the curve, ft/s^2,
$V_{exp,PC}$	=	expected speed at the PC, mph, and
R_{min}	=	minimum traveled path radius allowable to provide driver comfort, ft.

B. **Next, the lateral acceleration of a vehicle on the curve at the PT needs to be calculated.** To calculate this value, the expected speed at the PT should be substituted with the expected speed at the PC in Equation 8.13.

C. **Now that the vehicle's lateral acceleration on the curve at the PC and the PT has been calculated, the change in lateral acceleration on the curve can be determined.** This value is determined by subtracting the lateral acceleration on the curve at the PT from the lateral acceleration on the curve at the PC.

D. **The next value that needs to be calculated is the time to decelerate between the PC and PT.** This value can be calculated using the appropriate deceleration time equation from Table 7.2. Equation 8.14 displays the formula for the time to decelerate between the PC and PT.

$$T_{PC\rightarrow PT} = ((a)(\text{distance}_{PC\rightarrow Stop}) + b) - ((a)(\text{distance}_{PT\rightarrow Stop}) + b)$$
(8.14)

where,

$T_{PC\rightarrow PT}$	=	time to decelerate from PC to PT, sec,
distance $_{PC\rightarrow stop}$	=	distance from the PC to the stop, ft,
distance $_{PT\rightarrow stop}$	=	distance from the PT to the stop, ft,
a	=	slope of the linear equation from Table 7.2, dimensionless, and
b	=	intercept of the linear equation from Table 7.2, dimensionless.

E. **Now the rate of change in lateral acceleration on a curve can be determined and compared with the acceptable values in Table 7.3.** To calculate this value, the difference in lateral acceleration on the curve between the PC and PT should be divided by the time to decelerate from PC to PT. Equation 8.15 displays the formula for rate of change in lateral acceleration.

$$\text{Rate of } \Delta \text{ in lat acc} = \frac{\text{lat acc}_{PC} - \text{lat acc}_{PT}}{T_{PC \to PT}} \quad \textbf{(8.15)}$$

where,

Rate of Δ in lat acc	=	rate of change in lateral acceleration, ft/s^3,
lat acc$_{PC}$	=	lateral acceleration on the curve at the PC, ft/s^2,
lat acc$_{PT}$	=	lateral acceleration on the curve at the PT, ft/s^2, and
$T_{PC \to PT}$	=	time to decelerate from PC to PT, sec.

F. **Next, the value determined by Equation 8.15 must be compared to the desirable and acceptable values of 1-3 ft/s^3 and < 4 ft/s^3 respectively.** If the rate of change in lateral acceleration between the PC and PT is greater than 4 ft/s^3, then the process will need to start over and one or more design characteristic values will need to be changed. Refer to Appendix A to help determine which values may or may not need to be changed. If the rate of change in lateral acceleration between the PC and PT is less than 4 ft/s^3, then continue to the last design check.

G. **Lastly, the deceleration rate from the PT to the stop needs to be checked.** Equation 8.16 displays the formula for deceleration rate from PT to the stop.

$$\text{Decel Rate}_{PT \to stop} = \frac{\left(V_{stop}^{\ 2} - V_{exp,\,PT}^{\ 2}\right)\left(\frac{1}{3600}\right)}{\left(2 * \text{distance}_{PT \to stop}\right)\left(\frac{1}{5280}\right)} \quad \textbf{(8.16)}$$

where,

Decel Rate $_{PT \to stop}$	=	deceleration rate from PT to the stop, mph/sec,
V_{stop}	=	speed at the stop, 0 mph,
$V_{exp,PT}$	=	expected speed at the PT, mph, and
distance $_{PT \to stop}$	=	distance from the PT to the stop, ft.

H. **The deceleration rate from the PT to the stop should not exceed 7.6 mph/sec.** If the deceleration rate exceeds 7.6 mph/sec, one or more design characteristic values will need to be changed. If the criteria set forth in Table 7.3 are met, the design will be sufficient to accommodate most driver behaviors. Table 8.7 displays the details of each step.

Drainage Check for Minimum Transition Grades

The profile grade of the roadway alignment approaching the stop should be checked to assure that the grade used through the transitions at the PC and PT will provide adequate roadway drainage. Two techniques can be used to alleviate potential drainage problems, according to pages 190 and 191 of the 2001 Green Book (*4*):

1. Maintain minimum profile grade of 0.5 percent through the transition section, and
2. Maintain minimum edge of pavement grade of 0.2 percent (0.5 percent for curbed streets) through the transition.

Table 8.7. Procedure Step Details

Step No.	Procedure Step Description	Remarks
1	Select design speed for horizontal curve	Part A
	Select maximum superelevation rate at the PT end of the curve	Part B
	Select maximum side friction factor	Table 8.1, Figure 8.1, Part C
2	Calculate minimum radius	Equation 8.1, Part A
	Calculate tangent length	Equation 8.2, Part B
	Calculate length of curve (Lc)	Equation 8.3, Part C
	Calculate length of superelevation runoff at the PT	Equation 8.4, Table 8.2, 8.3, Part D
	Calculate length of tangent runout at the PT	Equation 8.5, Part E
	Calculate distance to transition from normal crown up/down to the crossroad pavement edge	Part F
	Calculate transition length	Equation 8.6
	Calculate distance from the PT to the crossroad edge	Equation 8.7, Part G
	Calculate distance from the PT to the stop	Part H
	Calculate distance from the PC to the stop	Part I
3	Calculate the expected speed at the PC	Table 7.1, Part A
	Iterate to find the maximum acceptable speed at the PC	Equations 8.8, 8.9, Part B
	Compare maximum acceptable speed at the PC to the expected speed at the PC	Part C
	Repeat until maximum acceptable speed at the PC is greater than expected speed at the PC	Part D
4	Calculate the expected speed at the PT	Table 7.1, Step 2, Part H
	Calculate the total available friction	Table 8.6, Part A
	Calculate the portion of friction available for braking	Equation 8.10, Part A
	Calculate braking distance	Equation 8.11, Part B
	Calculate minimum perception-reaction distance	Equation 8.12, Table 7.2, Part C
	Calculate perception-reaction plus braking distance	Part D
	Compare the perception-reaction/braking distance to the distance from the PT to the stop	Part E
5	Calculate the lateral acceleration on a curve at the PC	Equation 8.13, Part A
	Calculate the lateral acceleration on a curve at the PT	Equation 8.13, Part B
	Calculate the change in lateral acceleration on a curve	Part C
	Calculate the time to decelerate from PC to PT	Table 7.2, Equation 8.14, Part D
	Calculate the rate of change in lateral acceleration	Equation 8.15, Part E
	Compare the rate of change in lateral acceleration with the standard	Table 7.3, Part F
	Calculate the deceleration rate from the PT to the stop	Equation 8.16, Part G
	Compare the deceleration rate from the PT to the stop with the standard	Table 7.3, Part H
	Check for adequate drainage	2001 Green Book, pp 190,191

Example

Before the procedure can begin, several design characteristics must be known. For this example let:

Approach/Design Speed	=	65mph
Δ	=	15 degrees
Profile Grade of the Crossroad	=	+2 %
Profile Grade at the PT	=	-3%

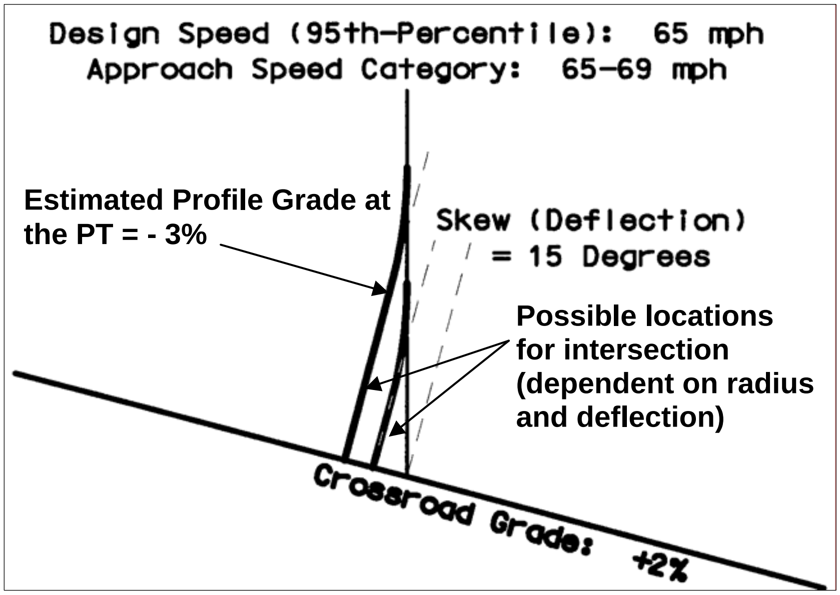

Figure 8.3 Situation Diagram of Example Intersection

Step 1

A. Let the assumed curve "design" speed be 30 mph.
B. Let the maximum superelevation at the PT end of the curve be 0.04 ft/ft or 4%.
C. Determine the maximum side friction factor (f_{max}) that corresponds to 30 mph. From Table 8.1, the f_{max} value can be determined by using the linear equation that falls between 24.85 and 31.07 mph. Equation 8.17 displays the linear equation that can be used to determine f_{max} at 30 mph.

$$f_{max} = -0.0061(30) + 0.404 \qquad \textbf{(8.17)}$$

The resultant f_{max} value is 0.221

Step 2

A. The minimum radius must be calculated.

$$R_{min} = 30^2/14.90((0.04) + 0.221) \qquad \textbf{(8.1)}$$

The minimum traveled path radius allowable to provide driver comfort is 231.43 ft.

B. Tangent length is the next geometric value that needs to be calculated.

$$T = 231.43 \tan\left(\frac{15}{2}\right) \qquad \textbf{(8.2)}$$

The tangent length is 30.47 ft.

C. The next item to calculate is length of the curve.

$$L_c = \frac{\pi(231.43)(15)}{180} \qquad \textbf{(8.3)}$$

The length of curve is 60.59 feet.

D. Now the length of superelevation runoff at the PT must be calculated. For this example, width of the roadway is 12 ft, n is 1, superelevation rate is 4% and b_w is 1. The maximum relative gradient for 30 mph is 0.66 from Table 8.3.

$$L_r = \frac{12(1)(4)(1)}{0.66} \qquad \textbf{(8.4)}$$

The length of superelevation runoff is 72.73 ft.

E. Next, the length of tangent runout at the PT must be calculated. For this example, superelevation of the normal crown ($e_{normal\ crown}$) is 2 percent.

$$\text{tan runout} = (2/4)(72.73) \qquad \textbf{(8.5)}$$

The length of tangent runout is 36.36 feet.
Next, multiply the length of tangent runout by 2 to determine the distance needed to transition from normal crown up or down to the crossroad pavement edge.

$$2\ (\text{tangent runout}) = 2\ (36.36)$$

The transition length to rotate from the adverse crown removed location to normal crown and back to the adverse crown removed location is 72.73 ft.

F. The next item to calculate is the length of transition to attain the crossroad grade which is +2% in this example.

$$\text{transition} = \frac{12(2)}{0.66} \qquad \textbf{(8.6)}$$

The transition length is 36.36 ft.

G. Next, the distance to the point of tangency from the crossroad pavement edge must be determined. Before this value can be calculated, the portion of the runoff located prior to the curve needs to be established. For 30 mph, and 1 rotated lane the portion of the runoff located prior to the curve is 0.80 according to Table 8.4

.

$$\text{Dist} = (36.36)(2) + (36.36) + (0.80)(72.73) \qquad \textbf{(8.7)}$$

The distance to the PT from the crossroad pavement edge is 167.27 ft.

H. Next, the distance from the PT to the stop must be calculated.

$$\text{Distance}_{\text{PT}\rightarrow\text{Stop}} = 167.27 - 10$$

The distance from the PT to the stop is 157.27 ft.

I. Now, the distance from the stop to the PC can be calculated.

$$\text{Distance}_{\text{PC}\rightarrow\text{Stop}} = 157.27 + 60.59$$

The distance from the PC to the stop is 217.86 ft.

Step 3

A. The expected speed at the PC must now be calculated using the speed profile equations in Table 7.1. For this example, the approach/design speed of the facility is 65 mph. Equation 8.18 displays the speed profile equation for the 69-65 mph category.

$$\text{Speed}_{\text{PC}} = 14.124 \text{ Ln } (217.86) - 40.347 \qquad \textbf{(8-18)}$$

The expected speed at the PC is 35.69 mph.

B. Now, the maximum superelevation at the PC must be assigned. For this procedure, the maximum superelevation at the PC is 0.06 ft/ft or 6%. Next, the linear equation that will complete the system of linear equations needs to be identified. Equation 8.17 displays the linear equation for 30 mph. The system of linear equations for this step is as follows:

$$V_{\text{max acc PC}} = \frac{f_{\text{max}} - 0.404}{-0.0061} \qquad \textbf{(8.8)}$$

$$V_{\text{max acc PC}} = \sqrt{231.43(15(0.06 + f_{\text{max}}))} \qquad \textbf{(8.9)}$$

The resultant maximum side friction factor is 0.22, which also results in a maximum acceptable speed at the PC of 30.84 mph.

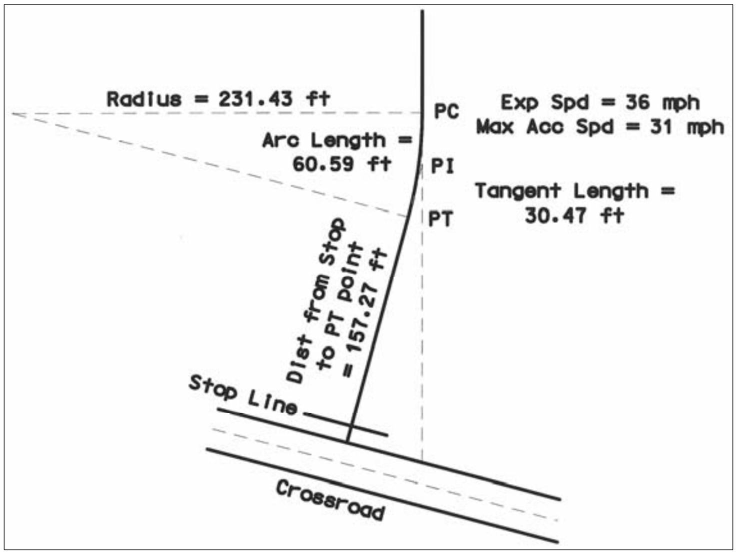

Figure 8.4 Situation Sketch Showing Calculated Curve Geometrics from Initial Curve "Design" Speed Selection

C. A comparison between the speeds shows that the expected speed at the PC is greater than the maximum acceptable speed at the PC. This means the process needs to start over from *Step 1*.

D. To proceed further, a speed for *Step 1* needs to be determined that will produce a maximum acceptable speed ($V_{max\ acc\ PC}$) greater that the expected speed at the PC.

For this example, choosing 35 mph results in a $V_{max\ acc\ PC}$ of 36.13 mph and an expected speed at the PC of 38.15 mph (difference = -2.02), which means the process must start over again.

Choosing 40 mph results in a $V_{max\ acc\ PC}$ of 43.16 mph and an expected speed at the PC of 40.70 mph (difference = 2.46). This choice is acceptable for moving to *Step 4*, but in order to minimize the design values to save on right-of-way and construction costs, the speed chosen in *Step 1* should produce the smallest difference possible between $V_{max\ acc\ PC}$ and expected speed at the PC where $V_{max\ acc\ PC}$ is greater that expected speed at the PC.

Choosing 37 mph for *Step 1* results in a $V_{max\ acc\ PC}$ of 38.21 mph and an expected speed at the PC of 39.24 mph (difference = -1.03), which means the process must start over again.

Choosing 38 mph results in a $V_{max\ acc\ PC}$ of 39.30 mph, and an expected speed at the PC of 39.63 mph (difference = -0.33), which means the process must start over again.

Choosing 39 mph results in a $V_{max\ acc\ PC}$ of 40.29 mph and an expected speed at the PC of 40.15 mph (difference = 0.15). Now that the speed at which $V_{max\ acc\ PC}$ is greater than the expected speed at the PC by the smallest margin has been determined, proceed to *Step 4*. The following geometric features are revised to conform to the new "design" speed of 39 mph:

- Maximum Side Friction Factor = -0.0037 (39) + 0.3239 = 0.1796
- R_{min} = 39^2/14.90(0.04 + 0.1796) = 464.85 ft,
- T = 464.85 tan (15/2) = 61.20 ft,
- L_C = 3.14 (464.85)(15)/180 = 121.70 ft,
- Distance from PT to the stop = {[2(81.36)/4] + [12(2)/0.59] + 0.80[(12)(1) (4)(1)/0.59]} - 10 = 177.13 ft
- Distance from PC to the stop = 177.13 + 121.70 = 298.83 ft

Step 4

A. The expected speed at the point of tangency must now be calculated. *Step 3* determined which speed profile equation is used to determine the expected speed at the PC. This equation can also be used to determine the expected speed at the PT. Equation 8.18 displays the formula.

$$\text{Speed}_{PT} = 14.124 \text{ Ln } (177.13) - 40.347 \qquad \textbf{(8.18)}$$

The expected speed at the PT is 32.77 mph.

B. Next, the stopping distance required for perception-reaction and braking must be calculated. First, the available friction for stopping at the PT needs to be calculated. To do this, the braking friction factor must be determined from Table 8.6 for a speed of 32.77 mph. The braking friction factor for 33 mph (32.77 mph rounded up) is 0.34. The maximum side friction factor for 39 mph (the "design speed of the curve) is calculated using Equation 8.19.

$$f_{max} = -0.0037(39) + 0.3239 \qquad \textbf{(8.19)}$$

The maximum side friction is 0.1796.

The available friction for stopping may now be determined.

$$f_B' = (0.34^2 - 0.1796^2) \qquad \textbf{(8.10)}$$

The available friction factor is 0.29 which makes the portion of total friction available for braking equal to 0.29/0.34 or 0.85.

Now the braking distance must be calculated. The grade at the PT for this problem is –3 %.

$$d = 32.77^2/30((11.2/32.2) - 0.03(0.270/0.34) \qquad \textbf{(8.11)}$$

The braking distance is 132.05 ft.

C. Now, the distance traveled during perception-reaction must calculated. This value can be determined by Equation 8.12, with a perception-reaction time of 1 second, and an expected speed at the PT of 32.77 mph.

$$\text{PR distance} = 1.47(32.77)(1) \tag{8.12}$$

The distance traveled during perception-reaction is 48.17 feet.

D. Now that the distance traveled during braking is known along with the distance traveled during perception-reaction, the total distance needed for stopping can be determined. When the two distances are added, the total distance is 180.22 ft.

E. Next, the distance from PT to the stop calculated in *Step 2, Part H* and the perception-reaction braking distance need to be compared. In this instance, the distance needed for perception-reaction braking (180.22 ft) is greater than the distance required for transition (177.13 ft). This means that vehicles may not have enough roadway distance to brake to a stop. Therefore, one of the design characteristics would need to be changed. The design characteristic that is changed depends on the user's preference. If it is feasible, the designer could choose a speed of 45 mph for *Step 1*. This would result in a design where the distance needed for transition is greater than the distance needed for perception-reaction braking. However, this choice will create a longer curve and take up more right-of-way. Another choice would be to reduce the grade at the PT. The braking distance equation is directly influenced by grade. If the grade is reduced, the amount of distance required for braking will be reduced. A change in grade at the PT would not affect the rest of the design up to this point. Several other design characteristics could be changed as well, such as deflection angle. Changing the deflection angle will change the entire design and the procedure will need to start over. If the crossroad profile grade is changed, it would affect the distance needed for transition. If the distance needed for transition is shorter than the distance needed for perception-reaction braking, the crossroad profile grade would need to increase in order for the transition distance to increase. Changing the profile grade of an adjacent or adjoining facility is almost always impractical and infeasible. The easiest of all these methods is changing the profile grade at the PT.

Step 5

A. Now that the design is done, the results can be evaluated according to the criteria in Table 7.3. First, calculate lateral acceleration on the curve at the PC.

$$\text{Accel on Curve}_{PC} = 40.15^2(5280^2/3600^2)/464.85 \tag{8.13}$$

The lateral acceleration on the curve at the PC is 7.46 ft/s^2.

B. Next, calculate the lateral acceleration on the curve at the PT.

$$\text{Accel on Curve}_{PT} = 32.77^2(5280^2/3600^2)/464.85 \tag{8.13}$$

The lateral acceleration on the curve at the PT is 4.97 ft/s^2.

C. The change in lateral acceleration on the curve can be determined by subtracting lateral acceleration on the curve at the PC from the lateral acceleration on the curve at the PT.

$$\text{Change in Acceleration} = 7.46 - 4.97$$

The change in lateral acceleration on the curve between the PC and PT is 2.49 ft/s^2.

D. Next, the time to decelerate between the PC and PT must be calculated. The time to decelerate from PC or PT can be found in Table 7.2. For this example, the equation corresponding to the approach speed category of 65-69 should be used. The new distances from the stop for the PC and PT for a 39 mph "design" speed are 298.83 ft and 177.13 ft respectively.

$$T_{PC\rightarrow PT} = ((0.0152)(298.83) + 2.336) - ((0.0152)(177.13) + 2.336) \quad \textbf{(8.14)}$$

The time to decelerate from the PC to the PT is 1.85 seconds.

E. Now the rate of change in lateral acceleration can be calculated.

$$\text{Rate of Change in Lat Acc} = (7.46 - 4.97)/1.85$$

The rate of change in lateral acceleration is 1.35 ft/s^3.

F. Once the rate of change in lateral acceleration is obtained, the value needs to be checked against the criteria set forth in Table 7.3. For this example the rate of change in lateral acceleration falls within the desirable range (1-3 ft/s^3).

G. Lastly, the deceleration rate from the PT to the stop needs to be checked.

$$\text{Decel Rate}_{PT\rightarrow Stop} = \frac{(0^2 - 32.77^2)(1/3600)}{(2)(177.13)(1/5280)} \quad \textbf{(8.16)}$$

The deceleration rate from the point of tangency to the stop is 4.45 mph/sec. This value falls below the maximum allowable value of 7.6 mph/sec. Now that all criteria have been checked, the procedure is complete.

Designer Aids for Easy Estimations

Chapter 9 contains graphical results of the procedure for several combinations of geometric elements. These results can help **estimate** what PT profile grade values will provide enough perception-reaction/braking distance to the driver as well as giving the designer an initial **estimate** of an appropriate radius with which to begin the procedure. For this example, the deflection is 15 degrees and the radius is 465 ft. Figure 8.5 displays the appropriate graph from Chapter 9.

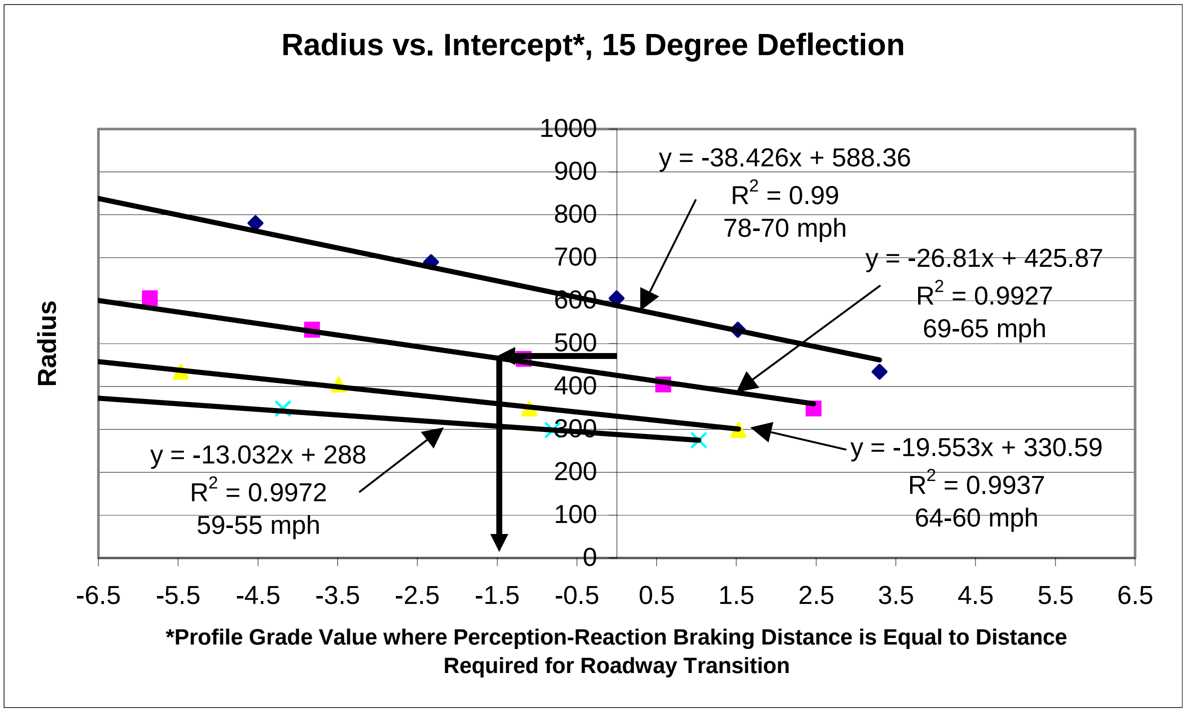

Figure 8.5. Radius vs. PT Grade for 15 Degrees Deflection

Tracing 465 ft on the radius axis to the 69-65 mph speed category line displays a PT grade intercept value of approximately –1.5%. The equation for the 69-65 mph speed category line estimates the PT grade intercept as –1.45% using a radius of 465 ft.

$$
\begin{aligned}
y &= -26.81x + 425.87 \\
465 &= -26.81x + 425.87 \\
x &= -1.45\%
\end{aligned}
$$

This means that PT down grades steeper than –1.45% may not give the driver a sufficient amount of roadway to brake to a stop if the vehicle is traveling at the "design" speed of the curve which is 39 mph. Conversely, PT down grades shallower than –1.45% or upgrades will provide the driver enough distance along roadway alignment to brake to a stop. Figures 8.6 and 8.7 display sketches of the example solution.

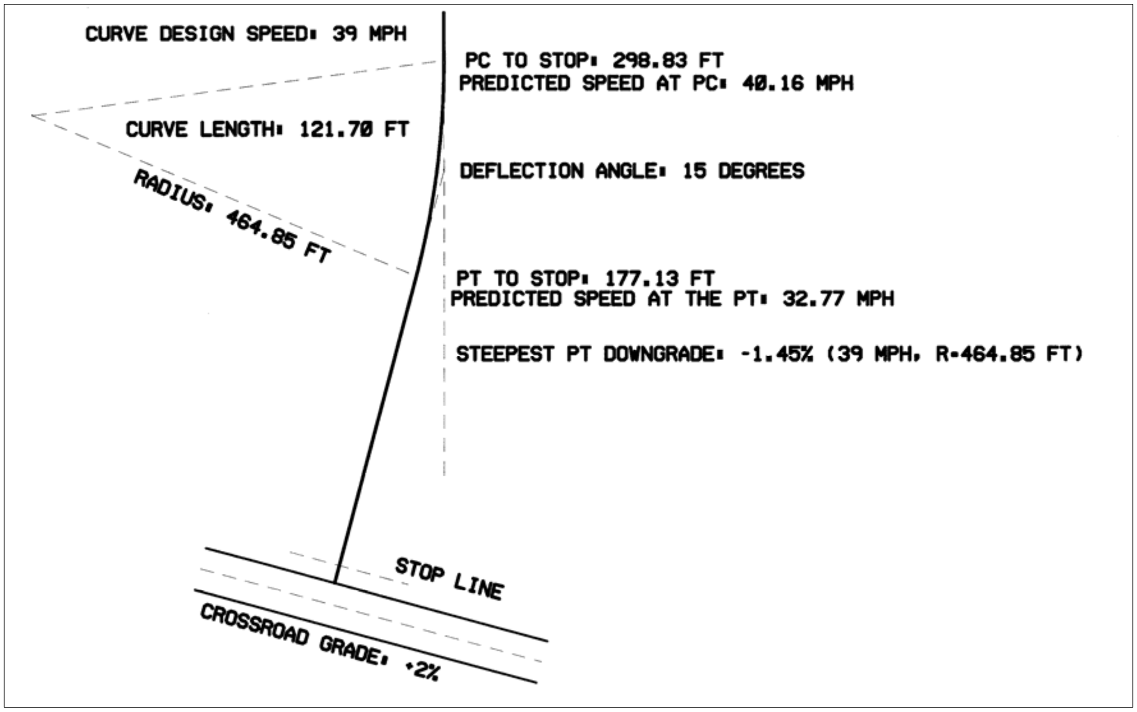

Figure 8.6. Sketch of the Example Solution

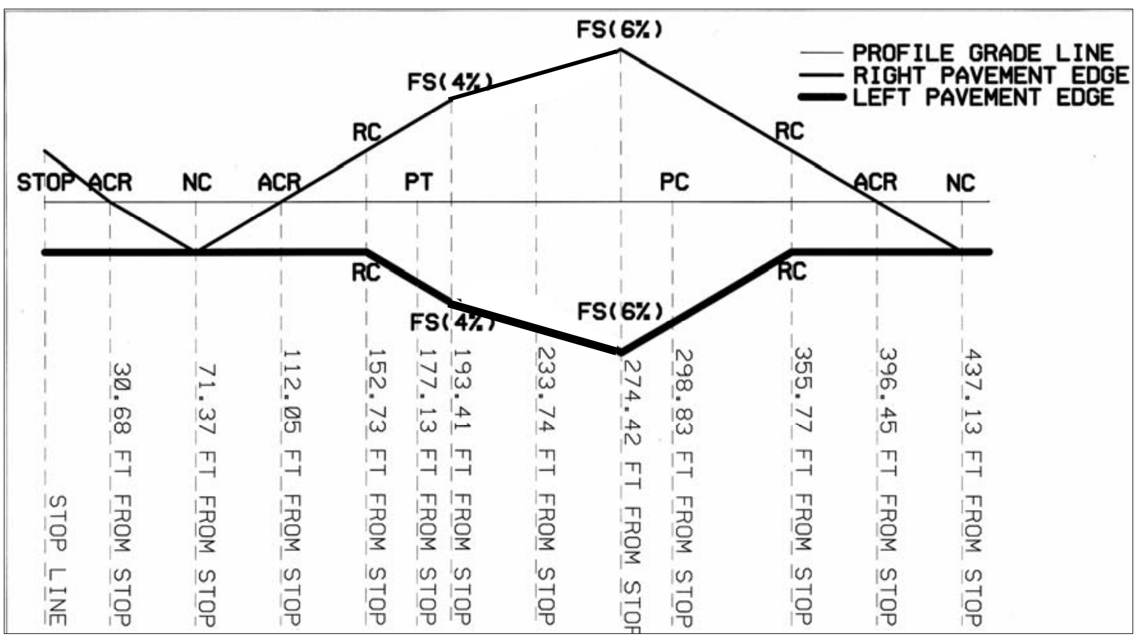

Figure 8.7 Superelevation Transition Diagram of the Example Solution

Drainage Check for Minimum Transition Grades

As recommended by pp. 190, 191 of the 2001 Green Book, the minimum profile grade should be at least ± 0.5% through the transition section and the minimum edge of pavement grade should be 0.2% for uncurbed roadways. To satisfy both of these criteria, the profile grade within the transition zone would have to be outside of the range of

–0.79% (-(max relative gradient of 0.59) – 0.2) and +0.79% (+(max relative gradient of 0.59) + 0.2). Therefore, appropriate grades for the conditions given in the example are between –1.45% and –0.79% and +0.79% and +6.5%.

Comparison of Procedure Solutions with Existing Study Site Curves

This research also attempted to determine how well the procedure modeled actual driver behavior. Four of the fifteen curve sites had similar geometric characteristics of those produced by the procedure, which allowed for comparisons between existing curve designs and those created by the procedure. Deflection, profile grade at the PT, crossroad grade, and approach speed from each of the four study sites were used in the procedure to create a new design. From the new design, new distances to the PC and PT as well as the predicted speeds at these points were calculated. Next, the speeds predicted by the procedure were compared with speed data collected at the four sites. The speed data collected at the detectors from the four sites was used to create a mean interpolated speed. The distance to the PC and PT resulting from the procedure determined between which two detectors the actual speed data would be interpolated. Once these detectors were identified, individual vehicles were tracked from one detector to the next. Each vehicle's speed was then interpolated between each detector. Once this was done for each vehicle, the mean of the interpolated speeds was calculated. The mean interpolated speed was then compared with the predicted speed using a t-test conducted at the 95% level of confidence. The test showed that the speeds for 3 of the 4 sites were statistically significantly different. In each instance, the predicted speed was greater than the mean interpolated speed which would be expected since the speed profiles used for the procedure represent 95th-percentile speeds. These results show that the design procedure creates horizontal curve alignments that are conservative. The results of these comparisons are displayed in Table 8.8. The only site where the speeds were not statistically significantly different was Highway 25 Southbound to Highway 23 (025sb23).

Table 8.8. Speed Comparison Results

Study Site Location	**025sb23**	**047sb23**	**084wb13**	**S-54Dsb12**
Detector Location from the intersection (ft)	400	750	1000	750
PC	351.51	703.1	551.85	654.61
Detector Location from the intersection (ft)	300	500	500	500
PC Predicted Speed (mph)	38.9	49.7	44.4	46.4
PC Interpolated speed from detectors (mph)	37.3	42.2	42.5	44.5
Detector Location from the intersection (ft)	200	200	300	300
PT	145.2	169.14	162	197.07
Detector Location from the intersection (ft)	100	100	100	10*
PT Predicted Speed (mph)	28.0	30.4	29.4	31.8
PT Interpolated speed from detectors (mph)	27.2	29.5	27.4	24.5

*-speed was assumed to be zero at the stop line

Next, the values of the existing geometric elements were compared with the geometric elements created by the procedure. The summary of these values is displayed

in Table 8.9. The results of this comparison show that for the most part the procedure results in smaller radii values than those of existing horizontal curves which would result in a reduced amount of right-of-way necessary to accommodate the horizontal alignment.
.

Table 8.9. Comparison of Geometric Elements

Study Site	**025sb23**		**047sb23**		**084wb13**		**S-54Dsb12**	
	existing	procedure	existing	procedure	existing	procedure	existing	procedure
Distance from PT to Stop (ft)	128.9	135.2	368.1	159.14	95.7	152	384.8	187.07
Curve Length (ft)	543.4	206.31	1230	533.96	615	389.85	760	457.54
Radius (ft)	1145.9	434.59	2291.8	831.35	954.9	605.34	1145.9	689.88

Chapter 9
DESIGNER AIDS FOR EASY ESTIMATIONS OF APPROPRIATE HORIZONTAL CURVATURE APPROACHING A STOP

Background

This chapter includes the graphical results of the procedure detailed in Chapter 8 for several combinations of geometric elements. These figures can be utilized to estimate which combinations of geometric elements will create a desirable design. The geometric elements that determine the feasibility of any design are speed category, central angle (deflection), profile grade value where perception-reaction/braking distance is equal to distance required for roadway transition (referred to as the intercept in the following figures), profile grade of the crossroad, and radius. It is important to understand why the profile grade at the point of tangency is relevant. The profile grade at the PT directly affects the distance needed for perception-reaction/braking distance. The perception reaction/braking distance cannot be greater than the distance needed for transition or else the driver may have an insufficient amount of roadway in which to stop before the intersection.

Because infinite combinations of these geometric elements exist, each design element was evaluated with several different discrete values. The values used for the central angle were 1, 10, 15, 30, 45, 50, and 60 degrees. The intercept profile grade values for the PT were determined by graphing the distances from the PT to the stop for perception-reaction/braking and transition. The geometric elements used to determine the intercept were crossroad profile grades, deflection, and the profile grade. The profile grade values for the PT that were used to determine the intercept PT grades were limited to a range of –6.5% to +6.5%. The 2002 NDOR Minimum Design Standards Manual establishes these limits (*22*). The crossroad grades used in this analysis were limited to a range between 0.001% and 6.5% since the procedure accounts for whether the crossroad grade is negative or positive. This means that the result for a crossroad grade of –1.57% is the same as the result for 1.57%. Therefore, only positive values up to 6.5% were evaluated. Table 9.1 displays results for a design scenario with 10 degree central angle, and 0.001% (essentially flat) crossroad grade. Figure 9.1 displays the graphical result of this scenario.

Table 9.1. Stopping Distance for $\Delta=10^o$, and Crossroad Grade of 0.001%

Deflection = 10 degrees					
PG of Intersecting Roadway (%)	PG of @ PT (%)	Calculated Speed		Distance from PT to a stop	
		PC (mph)	PT (mph)	Transition (ft)	PR+BD (t=1) (ft)
0.001	-6.5	33.79	28.37	125.02	157.53
	-6	33.79	28.37	125.02	155.52
	-5	33.79	28.37	125.02	151.69
	-4	33.79	28.37	125.02	148.12
	-3	33.79	28.37	125.02	144.77
	-2	33.79	28.37	125.02	141.63
	-1	33.79	28.37	125.02	138.67
	0.001	33.79	28.37	125.02	135.88
	1	33.79	28.37	125.02	133.25
	2	33.79	28.37	125.02	130.76
	3	33.79	28.37	125.02	128.41
	4	33.79	28.37	125.02	126.17
	5	33.79	28.37	125.02	124.05
	6	33.79	28.37	125.02	122.03
	6.5	33.79	28.37	125.02	121.06

PG: Profile Grade, PC: Point of Curvature, PT: Point of Tangency, PR: Perception-Reaction, BD: Braking Distance, t: Time

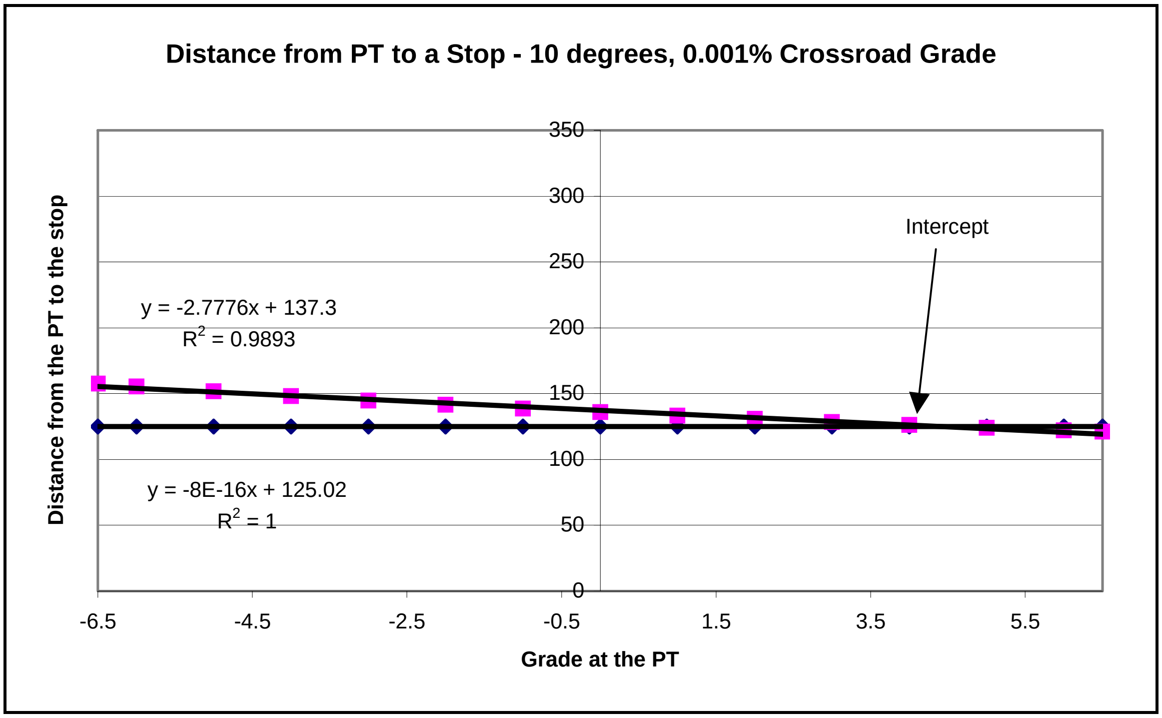

Figure 9.1. Graphical Results from Table 9.1

The constant line is the distance required for transition. It's slope is constant because the grade at the PT does not affect it. The sloped line represents the distances needed for perception-reaction/braking. These values are directly affected by the profile grade at the PT. Once the equation for each of the lines is determined, the intercept value can be calculated. For this scenario, the grade at which perception-reaction/braking distance and the distance required for transition intersect is 4.42%. This means that any PT grade less than 4.42% for this design scenario will not give the driver an adequate amount of roadway in which to stop. Intercept values were then used to create the graphical results.

Radius, Intercept and Deflection

The first useful relationship was developed between the radius, intercept, and deflection. Each graph can estimate the PT grades that will give the driver a sufficient amount of distance in which to brake to a stop before the intersection. First, determine the desired deflection and locate the graph closest to the desired deflection. The next step is to find the appropriate line for the desired approach speed category. Once the line is located, choose the desired radius for the design. Now that the radius and speed category have been determined, a corresponding intercept value can be determined. This corresponding intercept value represents an estimate for the lowest value the grade at the PT can be for the given conditions to provide an adequate amount of perception-reaction braking distance to the driver. For example, a horizontal alignment with design speed 60 mph, a 1 degree deflection and a radius of 200 ft estimates +2% as the lowest value the grade at the PT can be for the given conditions to provide an adequate amount of perception-reaction/braking distance to the driver. If a line is not visible on the graph, it means that any PT grade between –6.5% and +6.5% will provide the driver an adequate amount of perception-reaction/braking distance to the driver. It can also be concluded that any PT grade that is greater than the intercept value (to the left of the approach speed category line for the given radius) is acceptable, since it will reduce the distance required for perception-reaction/braking. Figures 9.2 to 9.8 display the results for the relationship between radius and intercept for a given deflection. Equations are also provided for each of the approach speed category lines.

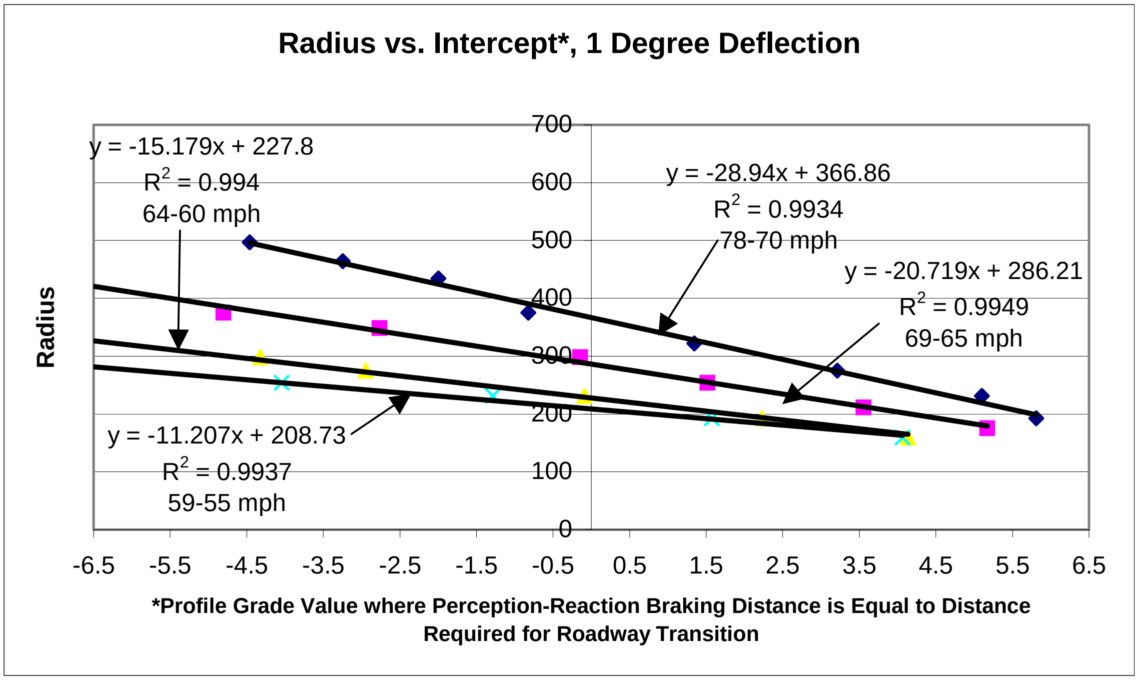

Figure 9.2. Graphical Results for Radius and Intercept with 1 Degree Deflection

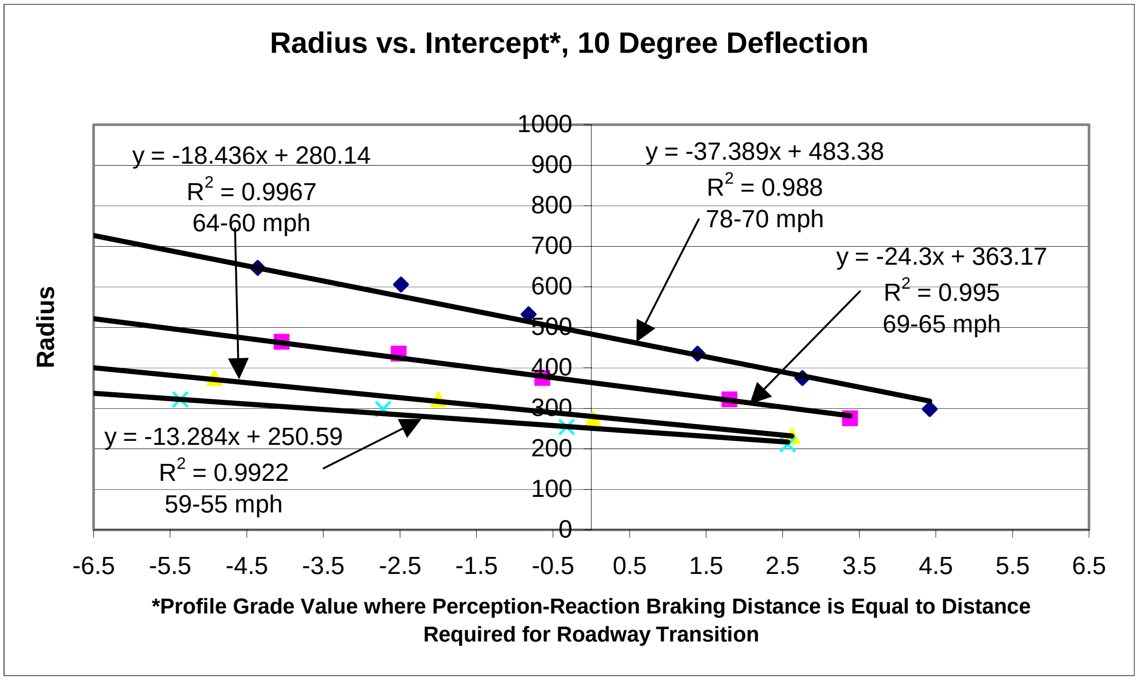

Figure 9.3. Graphical Results for Radius and Intercept with 10 Degree Deflection

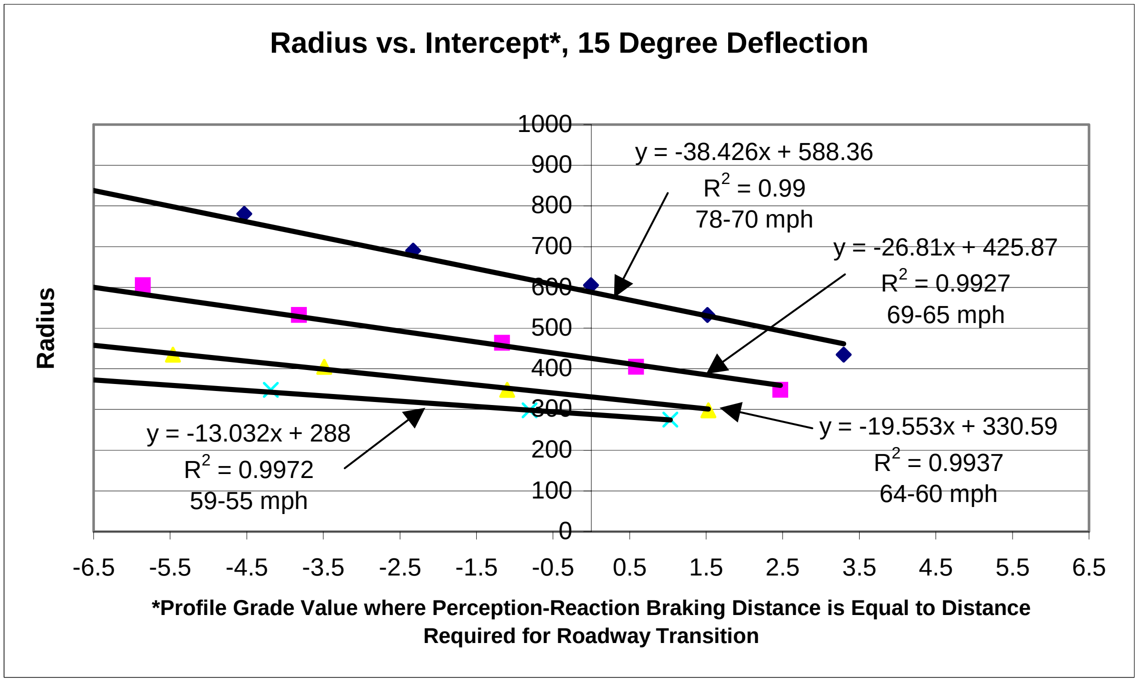

Figure 9.4. Graphical Results for Radius and Intercept with 15 Degree Deflection

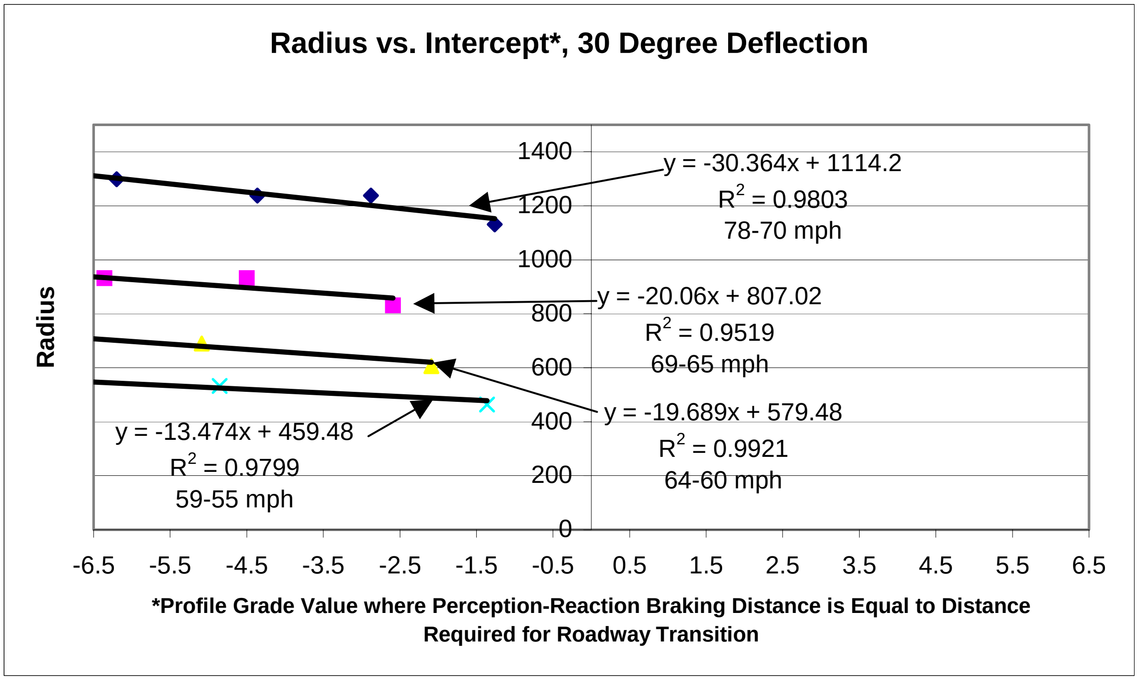

Figure 9.5. Graphical Results for Radius and Intercept with 30 Degree Deflection

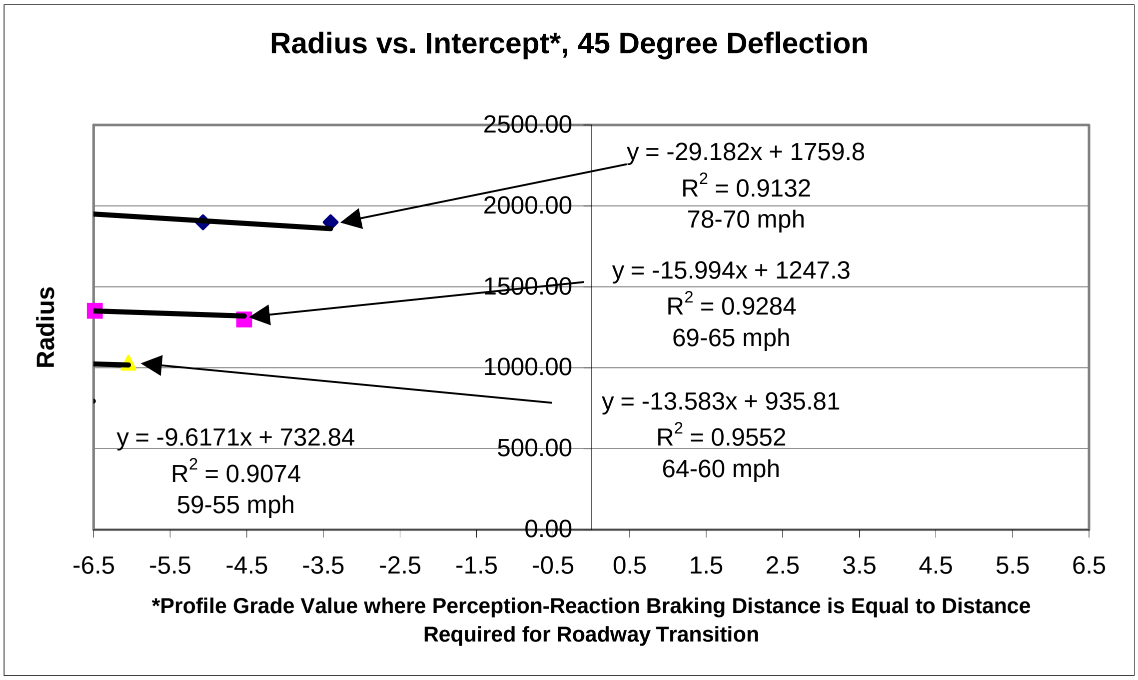

Figure 9.6. Graphical Results for Radius and Intercept with 45 Degree Deflection

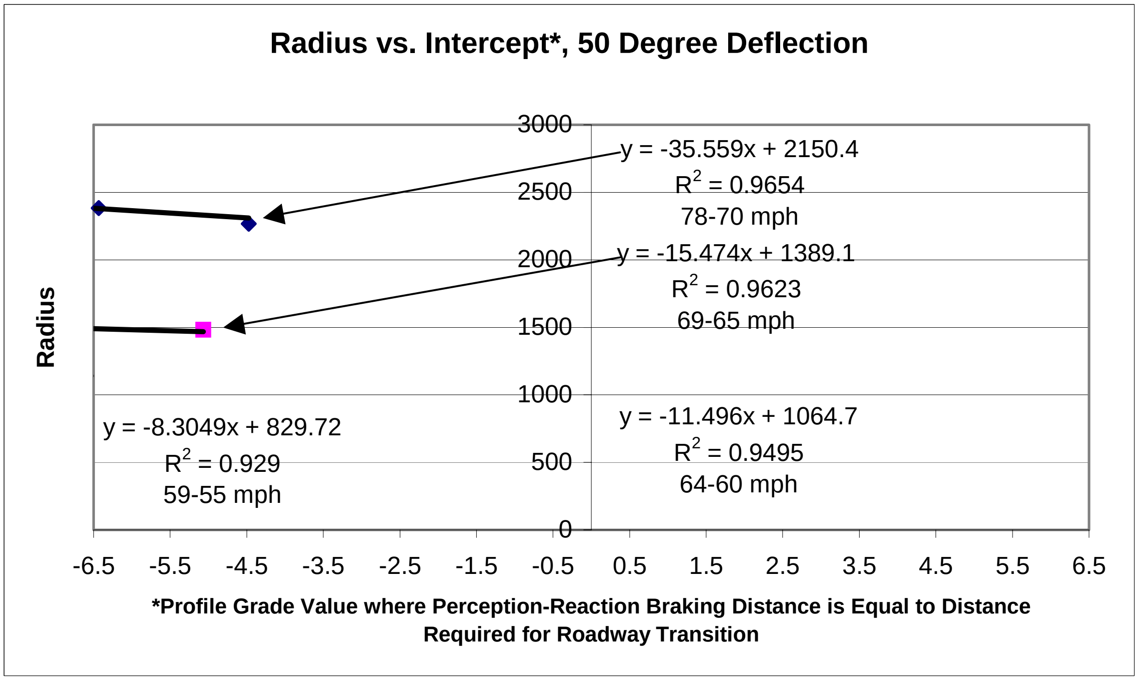

Figure 9.7. Graphical Results for Radius and Intercept with 50 Degree Deflection

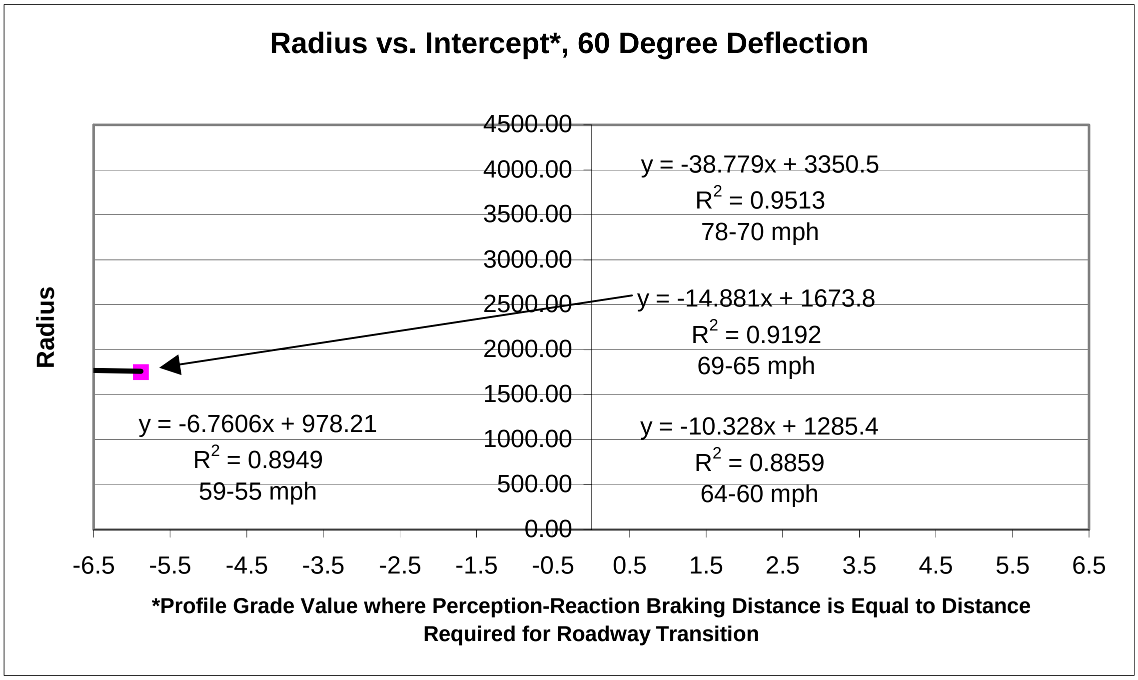

Figure 9.8. Graphical Results for Radius and Intercept with 60 Degree Deflection

Radius, Deflection and Crossroad Grade

The next relationship was developed between radius, defection and crossroad grade. This relationship can be used to estimate the radius for a given crossroad grade and deflection that will meet the criteria of the procedure. To begin, locate the graph with the crossroad grade that is closest to the desired or existing crossroad grade. Next, locate the regression line (or equation) that corresponds to the appropriate approach speed category. Once the line (or equation) is located, it can be used to estimate a radius that will create a suitable design for the desired deflection.

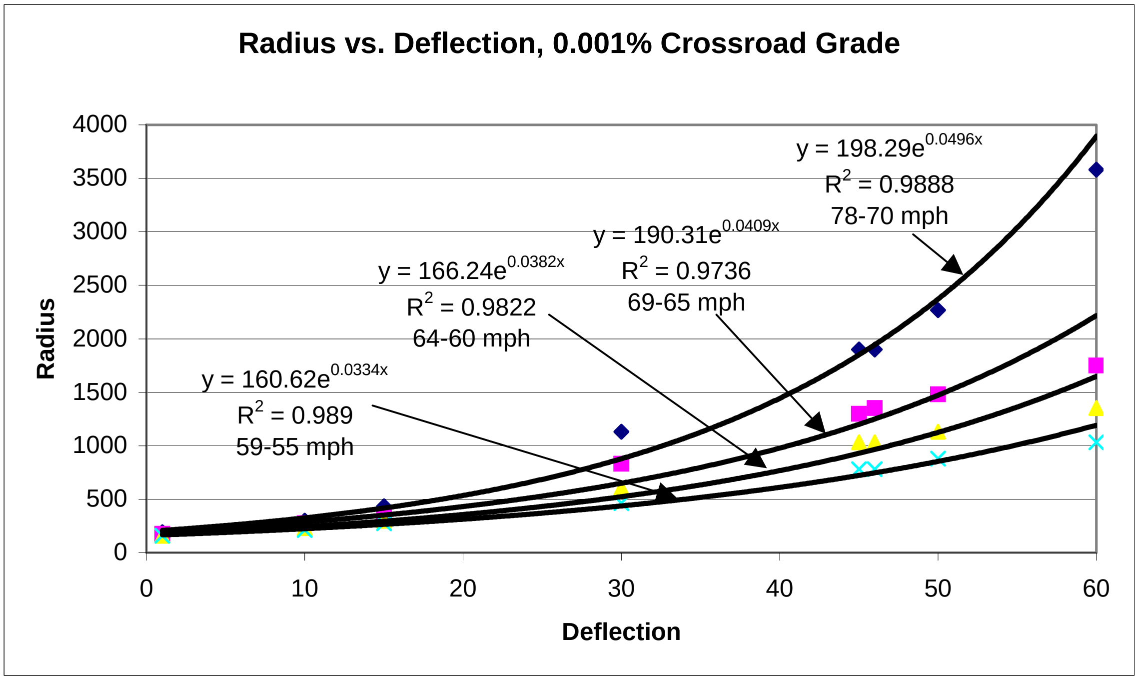

Figure 9.9. Graphical Results for Radius and Deflection with 0.001% Crossroad Grade

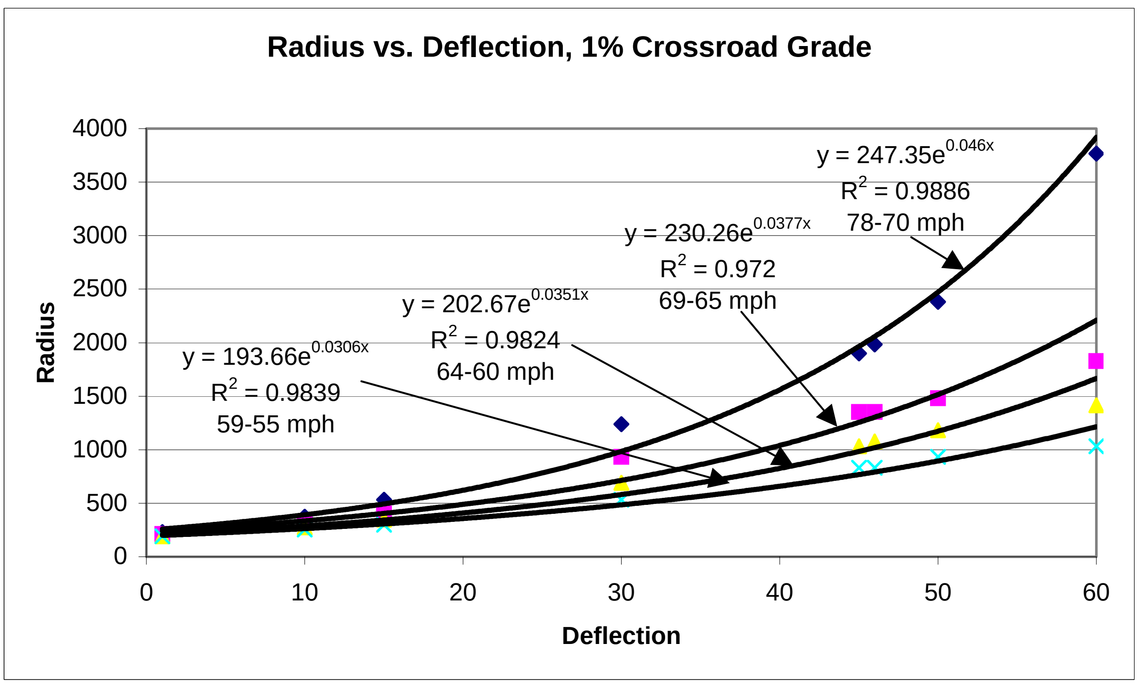

Figure 9.10. Graphical Results for Radius and Deflection with 1% Crossroad Grade

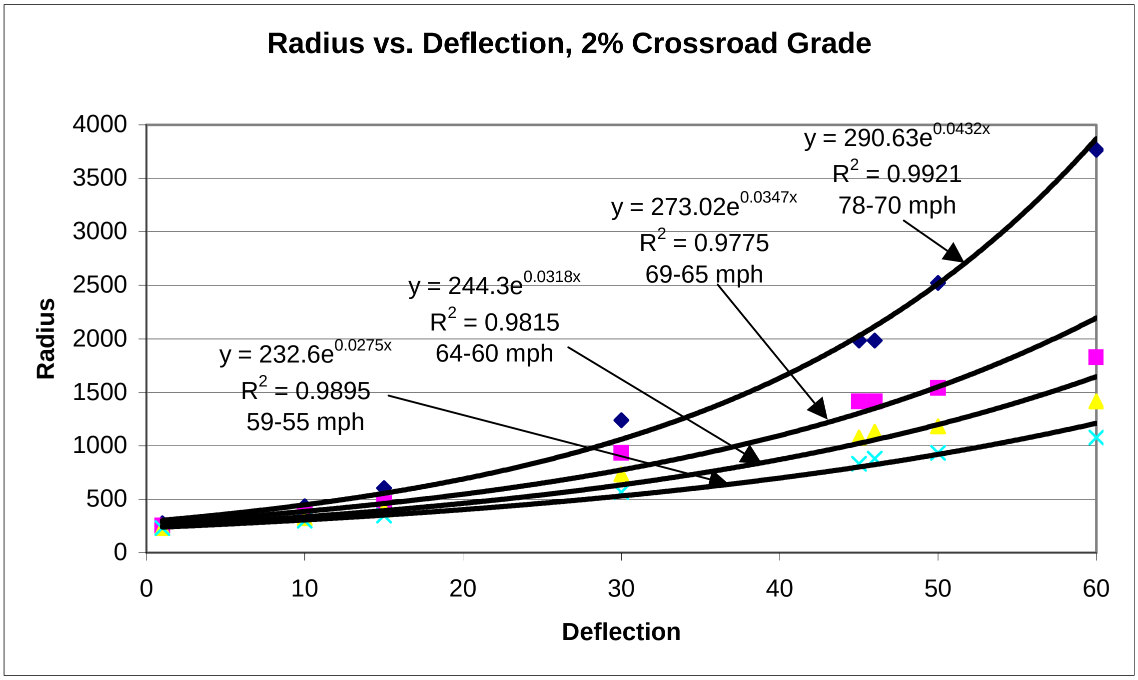

Figure 9.11. Graphical Results for Radius and Deflection with 2% Crossroad Grade

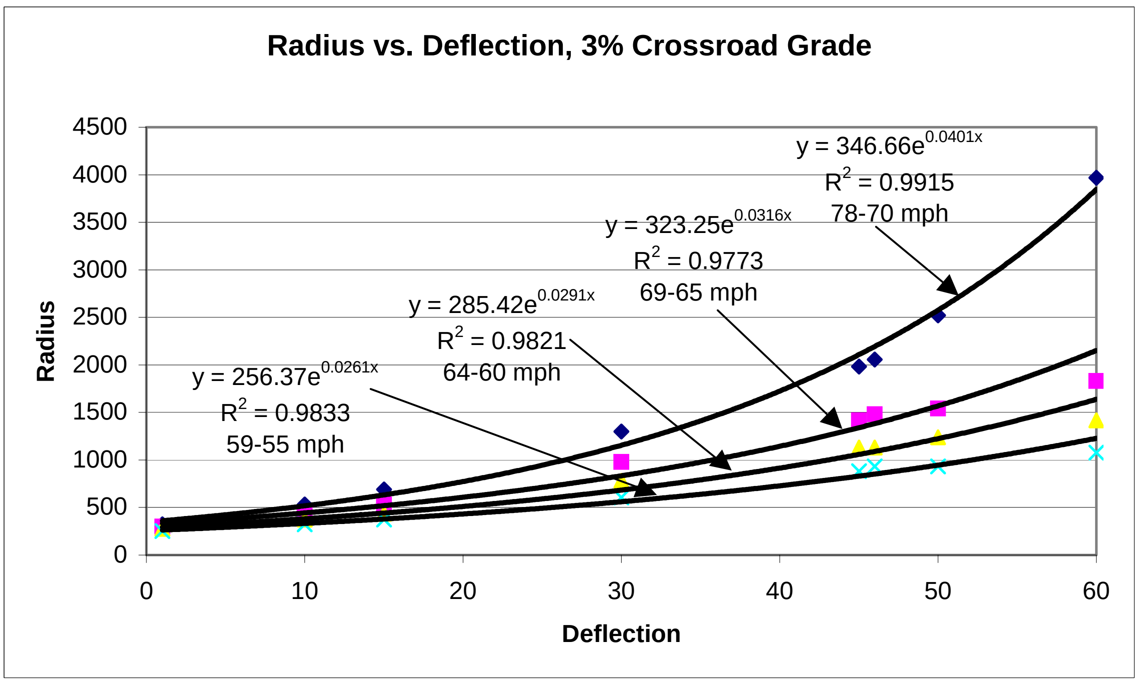

Figure 9.12. Graphical Results for Radius and Deflection with 3% Crossroad Grade

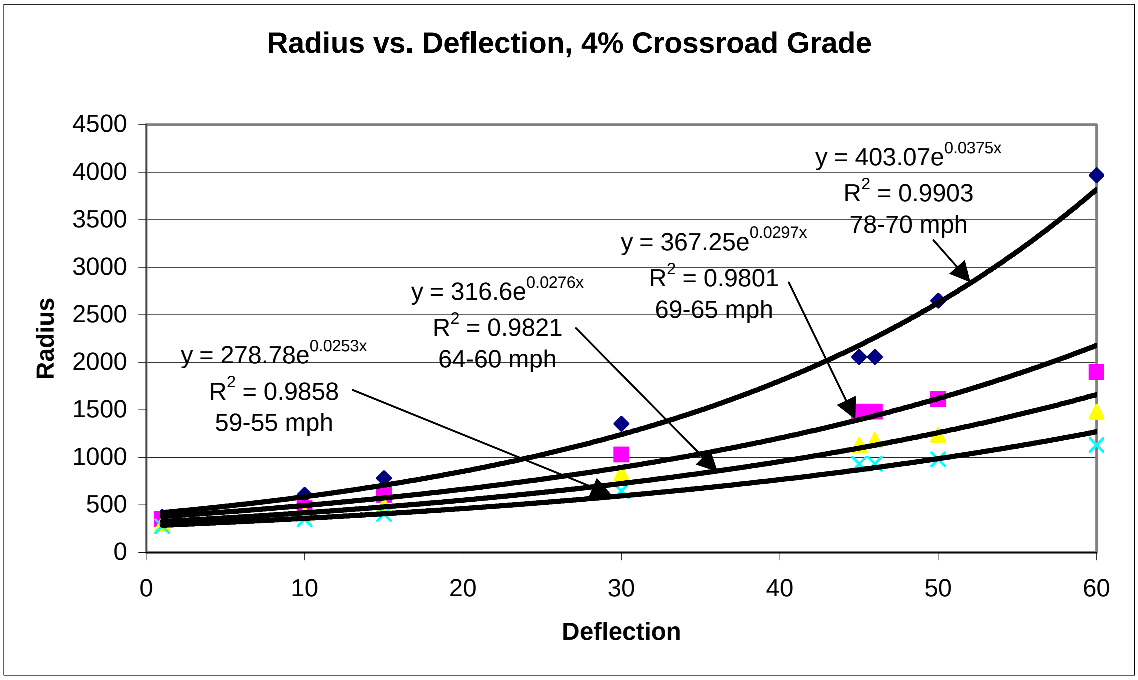

Figure 9.13. Graphical Results for Radius and Deflection with 4% Crossroad Grade

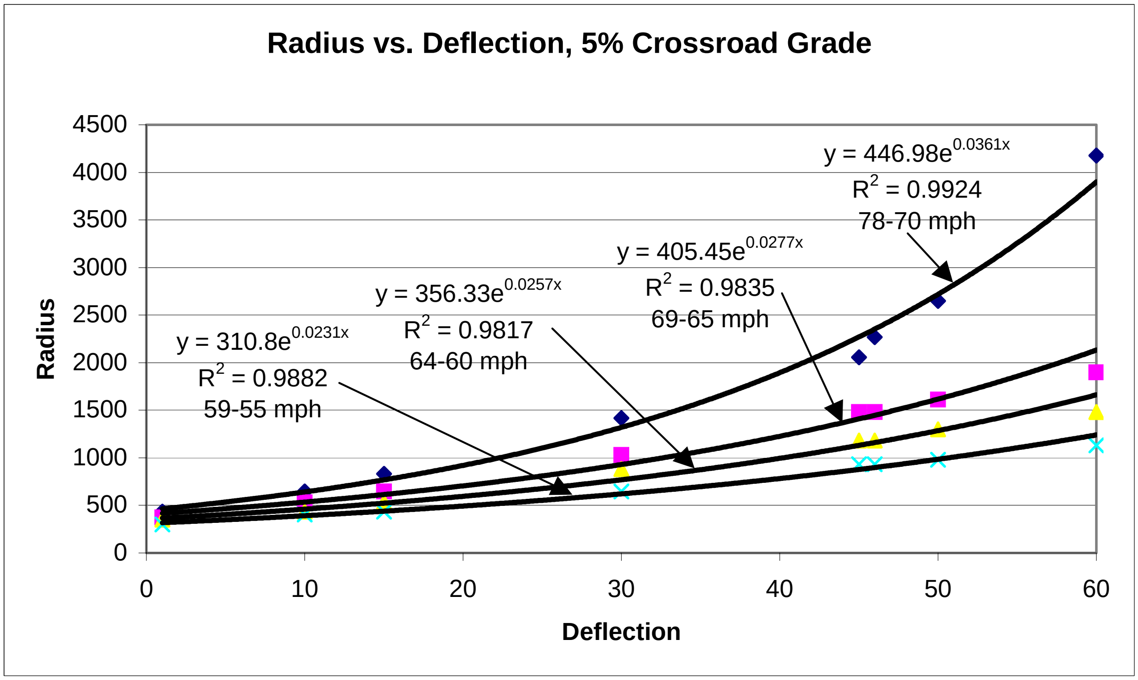

Figure 9.14. Graphical Results for Radius and Deflection with 5% Crossroad Grade

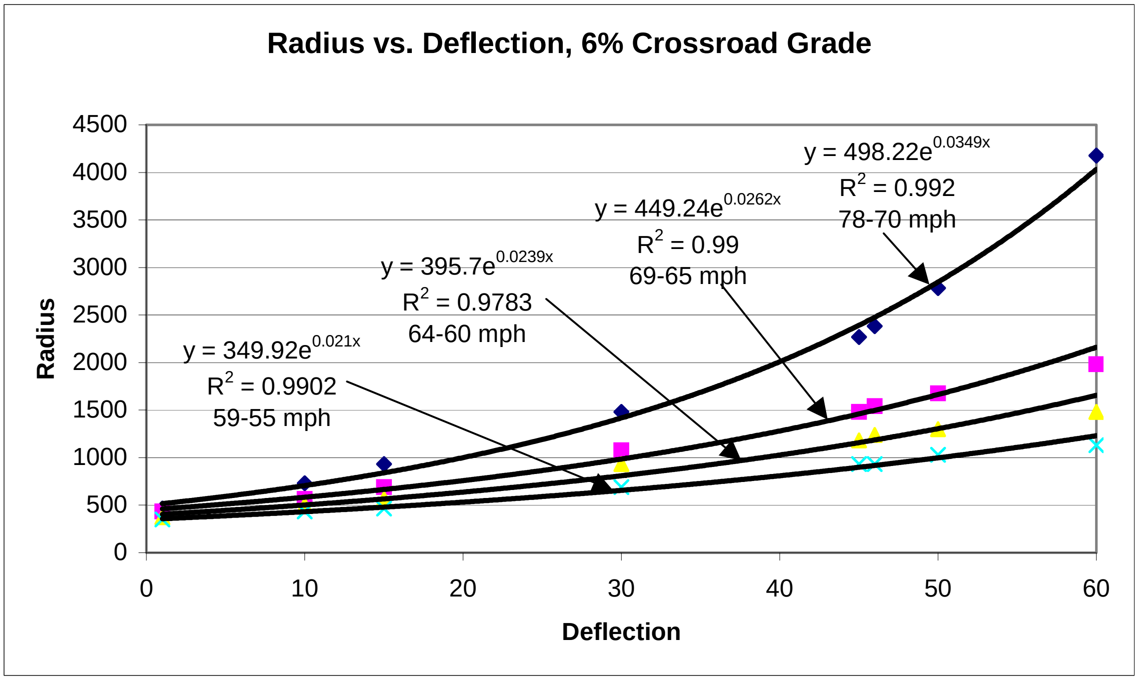

Figure 9.15. Graphical Results for Radius and Deflection with 6% Crossroad Grade

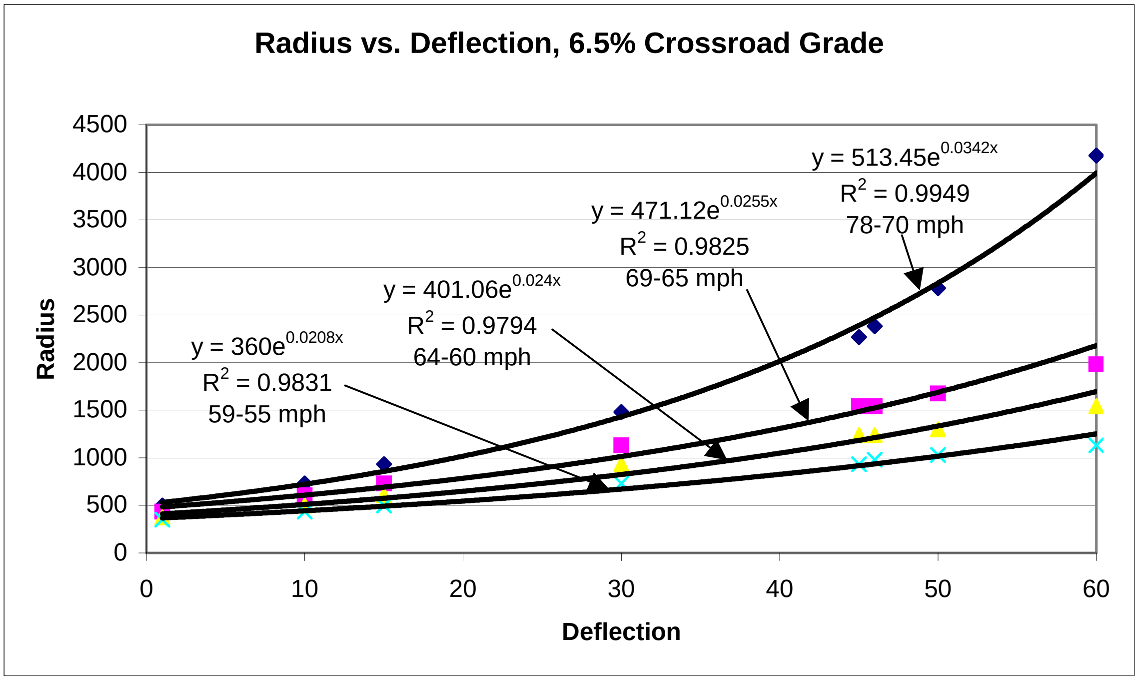

Figure 9.16. Graphical Results for Radius and Deflection with 6.5% Crossroad Grade

Chapter 10
CONCLUSIONS AND RECOMMENDATIONS

Conclusions

A procedure for the design of horizontal curves on two-lane, two-way rural highways approaching a stop-controlled intersection was developed in this research. Data from three tangent study sites were used to develop speed profile and deceleration time equations for use in the procedure. These data were limited to free-flow passenger cars during daylight hours under dry pavement conditions. Speed profiles and deceleration time equations were separated into four different categories by approach design speed or 95^{th}-percentile speed. This was done to reflect the range of speeds for approaching vehicles. The speed categories were 55-59 mph, 60-64 mph, 65-69 mph, and 70 mph and greater. Tables 7.1 and 7.2 display the equations developed for this procedure.

The developed design procedure can be used for all curve combinations (simple, compound and reverse) because it only focuses on the curve closest to the intersection. This distinction can be drawn because drivers will *maintain nearly the same speed* along curves prior to the curve closest to the intersection, while drivers on the curve closest to the intersection will begin to *decelerate significantly* as they approach the stop.

Certain combinations of design or geometric elements create alignments with features that are undesirable. One such feature is driver comfort. There are two ways the procedure measures driver comfort: 1) the rate of change in lateral acceleration and, 2) the deceleration rate from the point of tangency to the stop. Any combination of geometric features that creates a rate of change in lateral acceleration greater than 4 ft/s^3, and/or a deceleration rate from PT to the stop greater than 7.6 mph/s is considered undesirable and one or more design geometric elements may need to be changed.

Another feature is the amount of side friction and superelevation provided to the driver by the design procedure. A design with insufficient side friction and superelevation results when the maximum acceptable vehicle speed at the point of curvature is exceeded by the expected vehicle speed at the PC.

The distance required for perception-reaction and braking is another feature that is considered in the procedure. The design provides enough distance for perception-reaction and braking only if it is less than the distance needed for the road to transition out of the curve into the profile grade of the crossroad. If the perception-reaction/braking distance is greater, the driver may not have a desirable length of roadway between the point of tangency and the intersection to stop. Appendix C displays the graphical results of several combinations of geometric elements. These graphs can be utilized to easily estimate which combinations of geometric elements will create a desirable design.

The design procedure uses two different maximum side friction factors. The driver will be traveling faster at the point of curvature than at the point of tangency since the PC is further from the stop. For this reason, the maximum side friction factor used at the point of curvature will be higher than the one used at the point of tangency. The same reasoning applies to using a 6% maximum superelevation at the PC and a 4% superelevation maximum from near the midpoint of the curve through the PT.

If design procedures such as the one developed in this research were appropriate and available, the design of horizontal curves approaching a stop would become more uniform. A uniform or consistent design is desirable because it conforms to driver expectations. Research has found that if a road is consistent in design, then the road should not inhibit the ability of motorists to control their vehicle safely (*2*). Also, consistent roadway design should ensure that "most drivers would be able to operate safely at their desired speed along the entire alignment (*3*)."

Recommendations

Further research is recommended on this topic to determine whether the procedure is applicable and valid. A horizontal alignment using the proposed design procedure should be constructed. The project plans should contain enough detail of the superelevation transition to assure that field personnel will stake and construct the roadway according to the design procedure. Speed data should then be collected in a manner similar to this research. The data should be compared to the speed profile and deceleration curves developed and the procedure should be revised as necessary. Data for nighttime and wet conditions should be explored further for use in the development of speed profile and deceleration time equations.

Instructional Guidebook

Immediately following the references is an instructional guidebook to which roadway designers can refer for assistance when using the design method proposed in this report. The methodology is explained in condensed form and examples are used to clarify procedures and spreadsheet use.

REFERENCES

1. *Nebraska Department of Roads Roadway Design Manual*, Nebraska Department of Roads, December 1996.

2. Glennon, J.C., and D.W. Harwood. *Highway Design Consistency and Systematic Design Related to Highway Safety*. Transportation Research Record 681, Transportation Research Board, National Research Council, Washington, D.C., 1978, pp.77-88.

3. Lamm, R., E.M. Choueiri, B. Psarianos, and G. Soilemezoglou. *A Practical Safety Approach to Highway Geometric Design. International Case Studies: Germany, Greece, Lebanon, and the United States*. Paper presented at the International Symposium on Highway Geometric Design Practices, Boston, August 1995.

4. *A Policy on Geometric Design of Highway and Streets.* American Association of State Highway and Transportation Officials, Washington, D.C., 2001.

5. Harwood, D.W., and J.M. Mason, Jr. *Horizontal Curve Design for Passenger Cars and Trucks*. Transportation Research Record 1445. Transportation Research Board, National Research Council, Washington, D.C., 1994, pp. 22-33.

6. Fink, K.L., and R.A. Krammes. *Tangent Length and Sight Distance Effects on Accident Rates at Horizontal Curves on Rural Two-Lane Highways.* Transportation Research Record 1500. Transportation Research Board, National Research Council, Washington, D.C., 1995, pp. 162-168.

7. Anjus, V. and Maletin, M. *Speeds of Cars on Horizontal Curves.* Transportation Research Record 1612. Transportation Research Board, National Research Council, Washington, D.C., 1998, pp. 42-47.

8. http://terraserver.homeadvisor.msn.com

9. *State Highway Inventory Report*. Nebraska Department of Roads. 2000

10. Bonneson, J.A. *Superelevation Distribution Methods and Transition Designs*. National Cooperative Highway Research Program Report 439. Transportation Research Board, National Research Council, Washington, D.C., 2000.

11. *NU-METRICS HISTAR NC-97 Vehicle Magnetic Imaging Traffic Counter Operating Instructions.* NU-METRICS, Uniontown, PA. 1999.

12. Jessen, D.R. *Speed Prediction for Crest Vertical Curvature on Two-Lane Rural Highways in Nebraska.* University of Nebraska-Lincoln. 2000.

13. Antonucci, N.D., J.P. Tarris and J.M. Mason, Jr. *Accuracy and Visibility of Speed Measurement Devices.* Compendium of Technical Papers for the 66th Annual Meeting. Institute of Transportation Engineers, Washington, D.C. pp. 336-340.

14. http://www.weather.com.

15. *STATGRAPHICS Plus 5.0*, Statistical Graphics Corporation. 1994-2000.

16. Tarris, J.P., C.M. Poe, J.M. Mason, Jr., and K.G. Goulias. *Predicting Operating Speeds on Low-Speed Urban Streets: Regression and Panel Analysis Approaches*. Transportation Research Record 1523. Transportation Research Board, National Research Council, Washington, D.C., 1996, pp. 46-54.

17. Strickland, R. and H. McGee. *Evaluation of Prototype Automatic Truck Rollover Warning Systems*. Report No. FHWA-RD-97-124. Federal Highway Administration, Washington, D.C., 1998.

18. Woolridge, M., K. Fitzpatrick, R. Koppa, and K. Bauer. *Effects of Horizontal Curvature on Driver Visual Demand.* Texas Transportation Institute, Texas A& M University, College Station, Texas. Presentation Paper at 79th Annual Meeting, Transportation Research Board, January 2000.

19. Bonneson, J. *A Kinematic Approach to Horizontal Curve Transition Design.* Texas Transportation Institute, Texas A& M University, College Station, Texas. Presentation Paper at 79th Annual Meeting, Transportation Research Board, January 2000.

20. Fambro, D.B., K. Fitzpatrick, and R.J. Koppa. *Determination of Stopping Sight Distances.* National Cooperative Highway Research Program Report 400. Transportation Research Board, National Research Council, Washington, D.C., 1998.

21. Harwood, D.W., J.M. Mason, R.E. Brydia, M.T. Pietrucha, and G.L. Gittings. *Intersection Sight Distance.* National Cooperative Highway Research Program Report 383. Transportation Research Board, National Research Council, Washington, D.C., 1996.

22. *Nebraska Minimum Design Standards*, Nebraska Department of Roads, 2002.

23. Schurr, K., P.T. McCoy, D. Jessen, G. Pesti, and R. Huff. *Relationship Between Design, Operating, and Posted Speeds Under High-Posted Speed Conditions.* Nebraska Department of Roads Research Project Number SPR-PL-1(36) P519. Lincoln, NE. 2000.

24. Neuman, T.R., J.C. Glennon, and J.E. Leisch. *Functional Analysis of Stopping Sight Distance Requirements*. Transportation Research Record 923.

Transportation Research Board, National Research Council, Washington, D.C., 1983, pp. 59.

25. *A Policy on Geometric Design of Highway and Streets.* American Association of State Highway and Transportation Officials, Washington, D.C., 1990.

26. Halliday, D., R. Resnick. *Fundamentals of Physics*, John Wiley & Sons Publishing, 1974.