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7. Author(s)
Kay Fitzpatrick, Karl Zimmerman, Roger Bligh, Susan Chrysler, and Byron Blaschke

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The Texas A&M University System
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16. Abstract
The Texas Department of Transportation (TxDOT) is embarking on a multi-decade effort to expand the state's transportation system. To accomplish this expansion, TxDOT has expressed an interest in using higher design speeds (above 80 mph [130 km/h]) to promote faster and more efficient travel within the state. Current state and national roadway design guidance does not provide criteria for design speeds above 80 mph [130 km/h], so design values are not available. The purpose of TxDOT Project 0-5544, *Development of High Speed Roadway Design Criteria and Evaluation of Roadside Safety Features,* was to expand upon existing design guidance and identify new criteria for design speeds up to 100 mph [160 km/h]. Determination of preliminary criteria required extrapolation of existing equations along with the use of engineering judgment. A Roundtable Discussion Group was assembled to obtain practicing engineers' opinions and views on the methodology used to determine the criteria and on the specific values of the criteria. The technical report developed as part of this project presents issues and concerns and shows potential values generated for design speeds of 85 to 100 mph [140 to 160 km/h] for: stopping sight distance, grades, vertical alignment, lane width, shoulder width, cross slope, horizontal alignment and superelevation, ramp design speed, ramp grades and profiles, ramp cross section and cross slope, distance between successive ramps, ramp lane and shoulder widths, ramp acceleration and deceleration lengths, roadside clear zones, median width, roadside slopes and ditches, crash testing, and roadside safety devices. The criteria developed in the project were provided to TxDOT. TxDOT incorporated project findings in the new TxDOT *Roadway Design Manual* Chapter 8: Mobility Corridor (5R) Design Criteria.

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High Speeds, Controlling Criteria, Ramps, Roadside

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CRITERIA FOR HIGH DESIGN SPEED FACILITIES

by

Kay Fitzpatrick, Ph.D., P.E.
Research Engineer
Texas Transportation Institute

Karl Zimmerman, Ph.D., P.E.
Assistant Research Engineer
Texas Transportation Institute

Roger Bligh, Ph.D., P.E.
Research Engineer
Texas Transportation Institute

Susan Chrysler, Ph.D.
Research Scientist
Texas Transportation Institute

and

Byron Blaschke, P.E.
Research Engineer
Texas Transportation Institute

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TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

DISCLAIMER

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- **Maria Burke**, TxDOT Design Division, Project Director
- **Phil Russell**, TxDOT Texas Turnpike Authority (TTA), Program Coordinator
- **A. (Rory) Meza,** TxDOT Design Division, Project Advisor
- **David Bartz**, FHWA, Project Advisor
- **Marcy Saenz**, TxDOT Design Division, Project Advisor
- **Robert Stone**, TxDOT TTA, Project Advisor
- **Mark Blaschock**, TxDOT Bridge Division, Project Advisor

Participants in the April 26, 2006, Texas Roundtable Discussion Group included:

- **Bob Appleton**, Bryan District
- **Brian Barth,** Dallas District
- **David Bartz**, FHWA
- **Chuck Berry, Jr.,** El Paso District
- **Charles Conrad**, Ft. Worth District
- **Maria Burke**, TxDOT Design Division
- **James Koch,** Houston District
- **Mark Marek,** TxDOT Design Division
- **A. (Rory) Meza**, TxDOT Design Division
- **Reggie Richardson,** Waco District
- **Phil Russell**, TxDOT TTA
- **Marcy Saenz**, TxDOT Design Division
- **Clay Smith,** San Antonio District
- **Robert Stone**, TxDOT TTA
- **Robert Stuart,** Austin District

To take advantage of the international presence at the Transportation Research Board (TRB) Annual Meeting held in Washington, DC each January, the research team was able to hold an *International Roundtable Discussion on High Design Speeds* following the TRB Operational Effects of Geometric Design Committee Meeting (January 24, 2006, 5 to 7 pm). **Rich Cunard** of TRB and **Ray Krammes**, Chair of the Operational Effects of Geometric Design Committee, were instrumental in making the necessary arrangements for the meeting. The following individuals donated their time and energies in preparing and making presentations:

- Experiences and Proposals from Spain**, Alfredo Garcia,** Polytechnic University of Valencia
- Experiences from Germany, **Roland Weber,** Bast
- Overview of Italian Policy for High Speed Freeways**, Salvatore Cafiso,** University of Catania
- Experiences from United Kingdom, **John Smart**, Institution of Highways and Transportation

- Experiences from Australia, **Ricky Cox,** Queensland Department of Main Road**s**
- Experiences from Greece**, Basil Psarianos,** National Technical University of Athens

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CHAPTER 1

INTRODUCTION

The Texas Department of Transportation (TxDOT) is embarking on a multi-decade effort to expand the state's transportation system. This expansion includes the multiple, high-speed corridors of the Trans-Texas Corridor, as well as other facilities. TxDOT has expressed an interest in using high design speeds (above 80 mph [130 km/h]) for these facilities to promote faster and more efficient travel within the state. Currently, state and national roadway design guidance does not provide for design speeds above 80 mph [130 km/h], so roadway designers do not have the needed design values.

RESEARCH OBJECTIVES

The purpose of the Texas Department of Transportation Project 0-5544, *Development of High Speed Roadway Design Criteria and Evaluation of Roadside Safety Features* was to expand upon existing design guidance and identify new criteria for design speeds up to 100 mph [160 km/h]. This research report is a product of that project. The objective of the report is to discuss issues and concerns, along with generating potential values for design speeds of 85 mph [140 km/h] to 100 mph [160 km/h] for the controlling criteria, ramp design elements, and roadside items listed in Table 1-1.

RESEARCH APPROACH

Because limited information exists on design criteria for high speeds, the research team used a combination of existing information, extrapolations, and engineering judgment to develop preliminary criteria. Figure 1-1 shows a flowchart of the procedure used to achieve the objectives of the project.

The research efforts on this project began with a thorough search of the literature to determine existing knowledge. The search included reviews of current international, American Association of State Highway and Transportation Officials (AASHTO), and Texas practices along with historical policies. A traditional search of research studies and related literature was also conducted. These efforts revealed limited existing knowledge on driver performance at high speeds. Therefore, extrapolations of existing equations along with engineering judgments were used to determine preliminary criteria.

To obtain practicing engineers' opinions and views on the methodology used to determine the criteria and on the specific value of the criteria, a Roundtable Discussion Group was assembled. The Roundtable Discussion Group included representatives from:

- TxDOT's design division, districts, and turnpike authority;
- the Federal Highway Administration (FHWA); and
- the research team.

The Roundtable Discussion Group included 15 engineers along with four research team members. Prior to the Roundtable Discussion Group meeting, the research team distributed a Technical Memorandum detailing the research findings along with the preliminary criteria. During the meeting, the research team presented a very brief overview of each topic (generally less than 5 minutes) and then the group discussed the methodology used to determine the criteria and the proposed criteria. Table 1-1 lists the topics discussed. In general, most of the criteria as suggested by the research team were endorsed by the Roundtable participants. In a few cases, the Roundtable participants suggested additional investigations, which the research team conducted with the results forwarded to the TxDOT Design Division when completed.

Table 1-1. Topics Investigated As Part of Project 0-5544.

Controlling Criteria
• Stopping sight distance • Grades • Vertical alignment • Lane width • Shoulder width • Cross slope • Horizontal alignment and superelevation
Ramp Design Elements
• Ramp design speed • Ramp grades and profiles • Ramp cross section and cross slope • Distance between successive ramps • Ramp lane and shoulder widths • Ramp acceleration and deceleration lengths
Roadside Items
• Clear zones • Median width • Roadside slopes and ditches • Crash testing • Roadside safety devices

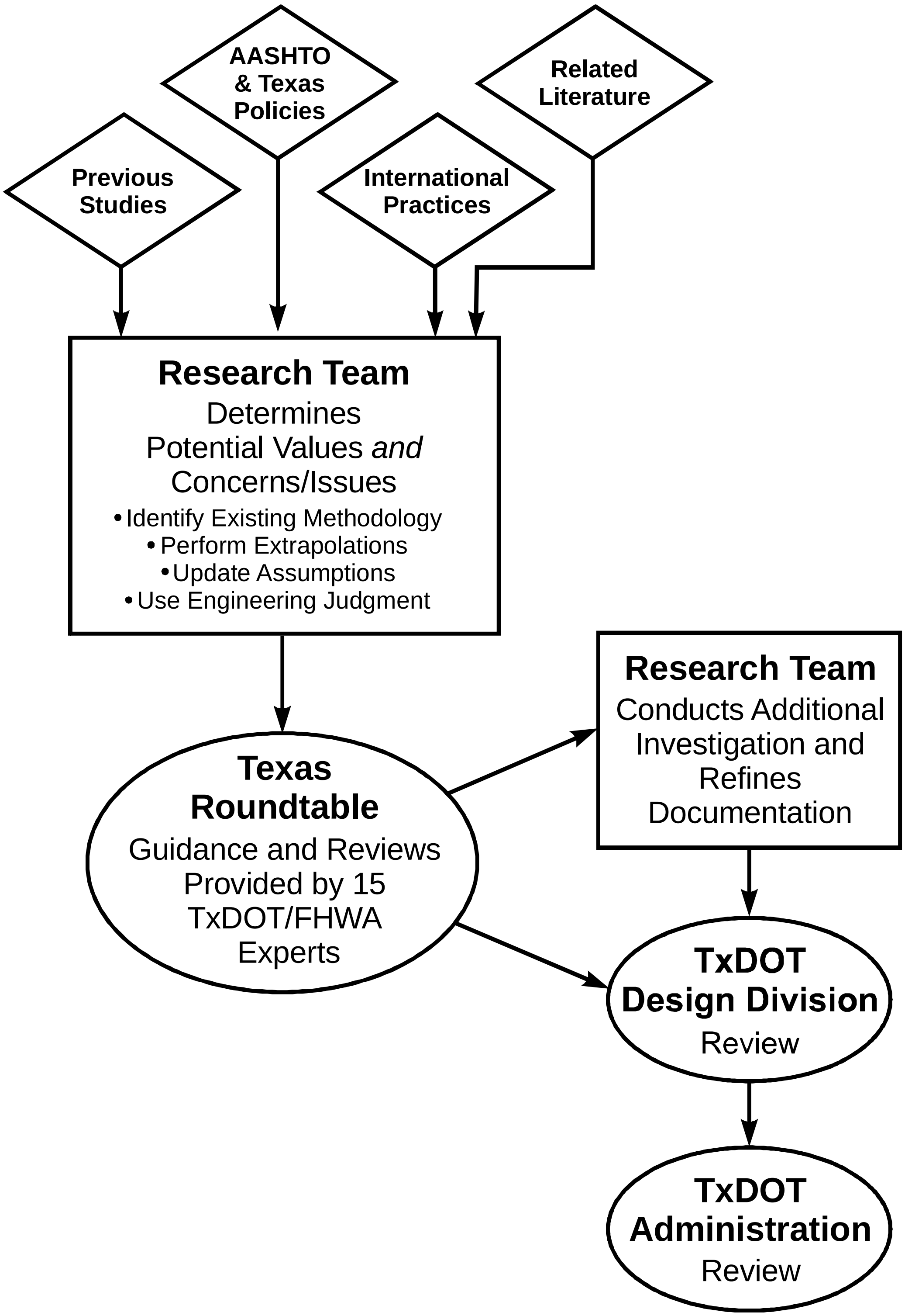


Figure 1-1. Flowchart of Research Approach.

REPORT ORGANIZATION

Investigations into each topic began by documenting existing knowledge, which was then used to begin the process of determining appropriate criteria for the higher speeds. Within the process of developing the higher speed criteria, gaps or concerns were identified along with the limitations of the current knowledge base. Researchers documented these gaps or concerns along with a list of research needs. Findings or comments deserving additional emphasis are bolded in this report.

Using the available knowledge, extrapolations, and engineering judgment, feasible design criteria values were determined. Shading is used in key tables to emphasize these potential values for high design speeds. For each chapter within this report covering one of the design topics (see Table 1-1), the following subheadings are typically used:

- Current Guidance,
- Other Guidance,
- Discussion,
- Potential Values for High Design Speeds, and
- Research Needs.

The chapters and their topics are:

- **Chapter 1 Introduction –** includes the objective of the project and the report organization.
- **Chapter 2 Stopping Sight Distance –** includes material on brake reaction time and driver visual capability along with other stopping sight distance elements.
- **Chapter 3 Grade –** provides discussion on critical length of grade including a sample of Texas speeds.
- **Chapter 4 Vertical Curves –** includes both sag and crest vertical curve material.
- **Chapter 5 Lane Width –** presents findings from lane positioning studies and truck studies.
- **Chapter 6 Shoulder Width –** discusses safety relationships with shoulder widths.
- **Chapter 7 Pavement Cross Slope** – reinforces the knowledge that hydroplaning is a key element with respect to pavement cross slope.
- **Chapter 8 Horizontal Alignment and Superelevation –** provides interesting findings with respect to friction and running speed.
- **Chapter 9 Ramp Design Speed –** presents discussion on limiting ramp design speed to an absolute difference between highway and ramp speed in addition to using the percent difference.
- **Chapter 10 Ramp Grades and Profiles –** provides information on grades for ramps.
- **Chapter 11 Ramp Cross Section and Cross Slope –** discusses superelevation and cross slope for ramps.
- **Chapter 12 Distance between Successive Ramps** - discusses the current guidance about ramp spacing and suggests that more work needs to be done in this area.

- Chapter 13 **Ramp Lane and Shoulder Widths –** provides information on widths for ramp lanes and shoulders.
- Chapter 14 **Acceleration and Deceleration Ramp Lengths –** uses extrapolation to determine suggested acceleration and deceleration ramp lengths along with adjustment factors for grades.
- Chapter 15 **Potential Deceleration and Acceleration Ramp Lengths for All Highway Speeds –** uses updated assumptions to determine new values. In most situations, current policies reflect recent knowledge; however, ramp acceleration and deceleration lengths along with adjustment factors for ramps have not been recently researched. There is information in the literature that could provide a better estimation of criteria than following the extrapolation methodology as was used in Chapter 14. Chapter 15 reflects the research team's initial thoughts and findings from a smaller scale effort where key assumptions were updated using information available in the literature.
- Chapter 16 **Roadside Clear Zones** – provides extrapolated values for clear zones for high design speeds. These values are based on vehicle steering and braking response instead of driver reaction time.
- Chapter 17 **Median Width** – provides extrapolated values for minimum median widths without barriers, as well as updated median barrier warrants. Like the clear zone values, the median width values are based on vehicle steering and braking response.
- Chapter 18 **Roadside Slopes and Ditches** – provides roadside slope values for high design speeds.
- Chapter 19 **Crash Testing** – discusses issues and challenges with current crash testing procedures and provides updated values for high speeds.
- Chapter 20 **Roadside Safety Devices** – describes the probable effectiveness of current safety hardware at higher speeds.

CHAPTER 2

STOPPING SIGHT DISTANCE

CURRENT GUIDANCE

Stopping sight distance (SSD) is the sum of the distance traversed during the brake reaction time and the distance to brake the vehicle to a stop. The following equation was used to generate the values included in the TxDOT *Roadway Design Manual* (TRDM) (*1*) and the 2004 *A Policy on Geometric Design of Highways and Streets* (commonly known as the *Green Book*) (*2*) (see Table 2-1 for a reprint of the values):

US Customary

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

Where:

- SSD = stopping sight distance, ft;
- V = design speed, mph;
- t = brake reaction time, assumed to be 2.5 s; and
- a = deceleration rate, assumed to be 11.2 ft/s^2.

Metric

$$SSD = 0.278Vt + 0.039\frac{V^2}{a} \qquad \textbf{(2-1)}$$

Where:

- SSD = stopping sight distance, m;
- V = design speed, km/h;
- t = brake reaction time, assumed to be 2.5 s; and
- a = deceleration rate, assumed to be 3.4 m/s^2.

Table 2-1. Stopping Sight Distance from TxDOT *Roadway Design Manual* (TRDM Table 2-1) and 2004 *Green Book* (GB Exhibit 3-1).

Design Speed (mph)	Brake Reaction Distance (ft)	Braking Distance on Level (ft)	Calculated Stopping Sight Distance (ft)	Design Stopping Sight Distance (ft)	Design Speed (km/h)	Brake Reaction Distance (m)	Braking Distance on Level (m)	Calculated Stopping Sight Distance (m)	Design Stopping Sight Distance (m)
15	55.1	21.6	76.7	80	20	13.9	4.6	18.5	20
20	73.5	38.4	111.9	115	30	20.9	10.3	31.2	35
25	91.9	60.0	151.9	155	40	27.8	18.4	46.2	50
30	110.3	86.4	196.7	200	50	34.8	28.7	63.5	65
35	128.6	117.6	246.2	250	60	41.7	41.3	83.0	85
40	147.0	153.6	300.6	305	70	48.7	56.2	104.9	105
45	165.4	194.4	359.8	360	80	55.6	73.4	129.0	130
50	183.8	240.0	423.8	425	90	62.6	92.9	155.5	160
55	202.1	290.3	492.4	495	100	69.5	114.7	184.2	185
60	220.5	345.5	566.0	570	110	76.5	138.8	215.3	220
65	238.9	405.5	644.4	645	120	83.4	165.2	248.6	250
70	257.3	470.3	727.6	730	130	90.4	193.8	284.2	285
75	275.6	539.9	815.5	820					
80	294.0	614.3	908.3	910					

When a highway is on a grade, the equation for stopping sight distance is modified as follows:

US Customary

$$SSD = 1.47Vt + \frac{V^2}{30\left(\left(\frac{a}{32.2}\right) \pm G\right)}$$

Where:

SSD = stopping sight distance, ft;
V = design speed, mph;
t = brake reaction time, assumed to be 2.5 s;
a = deceleration rate, assumed to be 11.2 ft/s^2; and
G = grade/100.

Metric

$$SSD = 0.278Vt + \frac{V^2}{254\left(\left(\frac{a}{9.81}\right) \pm G\right)} \qquad \textbf{(2-2)}$$

Where:

SSD = stopping sight distance, m;
V = design speed, km/h;
t = brake reaction time, assumed to be 2.5 s; and
a = deceleration rate, assumed to be 3.4 m/s^2; and
G = grade/100.

OTHER GUIDANCE

Minimum required stopping sight distances used by other countries *(3)* are shown in Table 2-2.

Table 2-2. Minimum Stopping Sight Distance and Brake-Reaction Time (T_{pr}) for Several Countries Collected in Early 1990s *(3)* and the 2004 *Green Book* Values *(2)*.

Country	Design or Operating Speed (km/h)													
	T_{pr} (s)	20	30	40	50	60	70	80	90	100	110	120	130	140
		Stopping Sight Distance (m)												
AASHTO (1994)	2.5	20	30	44	63	85	111	139	169	205	246	286		
Australia														
Normal Design	2.5	--	--	--	--	--	--	115	140	170	210	250	300	--
Normal Design	2.0	--	--	--	45	65	85	105	130	--	--	--	--	--
Restricted Design	1.5	--	--	--	40	55	70	--	--	--	--	--	--	--
Austria	2.0	--	--	35	50	70	90	120	--	185	--	275	--	380
Canada	2.5	--	--	45	65	85	110	140	170	200	220	240	--	--
France	2.0	15	25	35	50	65	85	105	130	160	--	--	--	--
Germany	2.0	--	--	--	--	65	85	110	140	170	210	255	--	--
Great Britain	2.0	--	--	--	70	90	120	--	--	215	--	295	--	--
Greece	2.0	--	--	--	--	65	85	110	140	170	205	245	--	--
South Africa	2.5	--	--	50	65	80	95	115	135	155	180	210	--	--
Sweden	2.0	--	35	--	70	--	165	--	--	--	195	--	--	--
Switzerland	2.0	--	--	35	50	70	95	120	150	195	230	280	--	--
AASHTO (2004)	2.5	20	35	50	65	85	105	130	160	185	220	250	285	--
-- = value not provided														

DISCUSSION

The key variables that affect the calculation of the stopping sight distance include:

- brake reaction time (also known as perception-reaction time), and
- deceleration rate.

Key variables associated with SSD include:

- driver eye height,
- object height,
- pavement friction, and
- driver visual capability.

Brake Reaction Time

Brake reaction time is the summation of perception time and brake time. Brake time was assumed as 1 s in 1940 and there have been no changes in the recommended value since then. Total brake reaction time ranged from 2 to 3 s, depending on design speed in previous editions of AASHTO policy. In 1954, the *Policies on Geometric Highway Design* (commonly called the *Blue Book*) (4) adopted a policy for total perception-reaction time of 2.5 s for all design speeds. However, the reason for the change could not be determined (5).

A mid-1990s National Cooperative Highway Research Program (NCHRP) project – NCHRP Report 400 (5) – provides information on driver braking performance to an unexpected object along with a summary of previous studies. Table 2-3 lists the mean estimates for various types of testing conditions identified in the literature as presented in NCHRP Report 400.

Table 2-3. Previous Reaction Time Findings as Reported in NCHRP Report 400 (5).

Type of Study, Number of Studies	Range in Number of Observations (Subjects)	Ages of Subjects	Mean Braking Time (s)	Standard Deviation (s)	Stimulus
Surprise (unsuspecting driver)					
Covert 4 studies	87 to 1644	Mix	1.27	0.66	Unexpected signal
Surprise 4 studies	15 to 1644	Young, old, and mix	1.28	0.20	Unexpected object or light
Anticipated (alerted driver) – to convert to surprise, correction factors of either 1.35 or 1.75 have been reported in literature.					
Driving simulator 4 studies	38 to 114	Old, mix	0.56 (converted to surprise = 0.76 to 0.98)	0.10	Onset red or Bumpa-Tel test
Behind the wheel 3 studies	15 to 321	Young, old, mix	0.73 (converted to surprise = 0.99 to 1.28)	0.16	Anticipated object or horn

As part of NCHRP Report 400, several studies were performed. Table 2-4 lists testing conditions and findings. In-vehicle instrumentation measured driver perception-brake reaction times (P-B RT), braking distances, and deceleration to unexpected and anticipated stops (5).

Table 2-4. NCHRP Report 400 Field Studies (5).

Study	Location	Vehicle	Subjects (Number)	Encountered	Initial Speed (mph)	Age of Subjects	P-B RT (s)	Std Dev (s)
1A	Closed	TTI	TTI (6)	Planned/ Surprise	40 and 55	Not reported	0.34	0.173
1B	Closed	TTI	TTI (3 expert drivers)	Planned/ Surprise	70	Not reported		
2	Closed	TTI	Pool (12)	Unexpected	55	Older	0.82	0.159
2	Closed	TTI	Pool (10)	Unexpected	55	Younger	0.82	0.203
3	Closed	Personal	Pool (7)	Unexpected	55	Older	1.14	0.353
3	Closed	Personal	Pool (3)	Unexpected	55	Younger	0.93	0.191
4	Open-road	Personal	Pool (5)	Unexpected	45	Older	1.06	0.222
4	Open-road	Personal	Pool (6)	Unexpected	45	Younger	1.14	0.204

Note: 1 mph = 1.61 km/h. TTI = Texas Transportation Institute.

For the nine drivers participating in Study 1, the overall perception-brake reaction time was 0.34 s, with a standard deviation of 0.173 s. The authors reported that neither the main effects of stopping condition (planned, surprise, or no signal), speed (40, 55, or 70 mph [65, 88, or 112 km/h]), or the interaction of condition and speed was statistically significant for reaction time. **Stated in another manner, for the nine TTI drivers the perception-brake reaction time did not vary by initial speed.** Note also that these nine drivers had perception-brake reaction times much less than those found for other participants (e.g., 0.34 s versus 0.93 s in Study 3).

While the perception-brake reaction times were not statistically different, the deceleration results for the participants in Study 1 were different for a 40 mph [65 km/h] initial speed versus a 55 mph [88 km/h] initial speed. Therefore Studies 2 and 3 used the 55 mph [88 km/h] initial speed. For the unexpected condition, the tests were performed on dry pavement with a tangent alignment. (The expected test condition was also performed on wet pavement and curves. However, only the unexpected results are presented in Table 2-4).

NCHRP Report 400 concluded that a mean perception-brake reaction time to an unexpected object scenario under controlled and open-road conditions is about 1.1 s (5). **The 95th percentile perception-brake reaction time for these same conditions was 2.0 s.** These results used findings from tests conducted with a 55 mph [88 km/h] initial speed. The findings from the NCHRP 400 work were consistent with those in the literature that state that most drivers are capable of responding to a stopping sight distance situation (i.e., an unexpected object in the roadway) within 2.0 s. Thus, AASHTO's brake reaction time of 2.5 s encompasses most of the driving population.

Deceleration and Pavement Friction

The NCHRP Report 400 research along with other studies showed that over 90 percent of drivers choose deceleration greater than 11.2 ft/s^2 [3.4 m/s^2] when confronted with the need to stop for an unexpected object in the roadway. This deceleration is within drivers' capabilities to stay within their lane and maintain steering control during the braking maneuver on wet surfaces. Therefore, it was recommended for the stopping sight distance procedure.

The portion of the NCHRP Report 400 study that included the nine TTI drivers that started the braking tests at 40, 55, or 70 mph [65, 88, or 112 km/h] found a constant deceleration value that ranged between 14.8 and 22.5 ft/s^2 [4.5 and 6.9 m/s^2]. The constant deceleration value was determined from the braking distance and the initial speed using the following:

US Customary

$$a = \frac{(1.47V)^2}{2BD} \qquad g = \frac{a}{32.2}$$

Where:

a = deceleration rate, ft/s^2;
V = initial speed, mph;
BD = braking distance, ft; and
g = equivalent constant deceleration.

Metric

$$a = \frac{(0.278V)^2}{2BD} \qquad g = \frac{a}{9.81} \qquad \textbf{(2-3)}$$

Where:

a = deceleration rate, m/s^2;
V = initial speed, km/h;
BD = braking distance, m; and
g = equivalent constant deceleration.

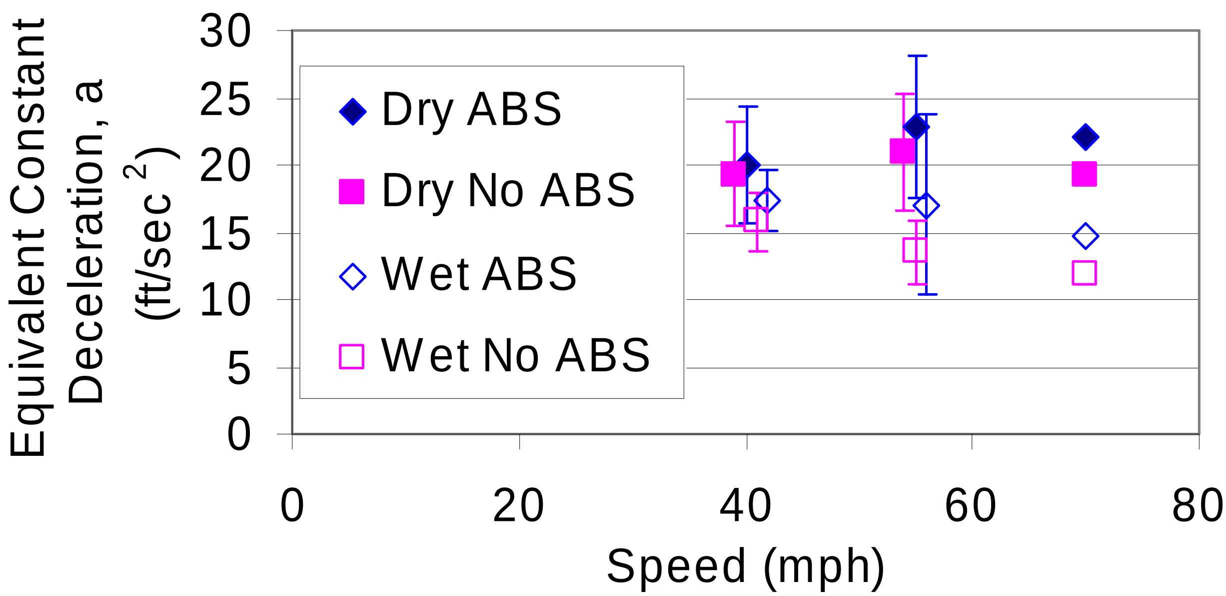
Table 2-5 lists the results in units of gravitational deceleration (g) as provided in NCHRP Report 400 for the runs with initial speeds of either 40 or 55 mph [65 or 88 km/h] for the nine TTI drivers. The 70 mph [112 km/h] data were not reported in a similar manner in NCHRP Report 400 because "the major interest was on the 40 and 55 mph [65 or 88 km/h] data." The mean equivalent constant deceleration values for the 70 mph [112 km/h] runs were calculated from braking distances provided elsewhere in the report. A visual representation of the deceleration data (in ft/s^2 as converted from g) is shown in Figure 2-1. Note that the data were spread out on the graphic so that the ranges could be seen – all runs were started at a consistent speed (either 40, 55, or 70 mph [65, 88, or 112 km/h]). The vertical bars for the 40- and 55-mph [65 or 88 km/h] data represent one standard deviation from the mean. For 40 mph [65 km/h], the use of an antilock brake system (ABS) was not significant, while wet or dry pavement conditions were significant at both 40 and 55 mph [65 and 88 km/h]. There were also statistically significant differences among drivers (5).

The report did not comment on whether there was a statistically significant difference in deceleration for different initial speeds. The visual review of the available data shows a potential downward trend in deceleration at higher initial speeds for the runs conducted on wet pavement. Additional tests would be needed to determine if even lower deceleration rates would be used by drivers at high initial speeds.

Table 2-5. Equivalent Constant Deceleration Based on NCHRP Report 400 Data (5).

Speed (mph)	Pavement	ABS	Number of Test Runs	g Mean	g Standard Deviation	a (ft/s^2) Mean	a (ft/s^2) Standard Deviation
40	Dry	No	191	0.60	0.122	19.3	3.9
40	Dry	Yes	176	0.62	0.134	20.0	4.3
40	Wet	No	203	0.49	0.067	15.8	2.2
40	Wet	Yes	186	0.54	0.071	17.4	2.3
55	Dry	No	216	0.65	0.135	20.9	4.3
55	Dry	Yes	203	0.71	0.163	22.9	5.2
55	Wet	No	146	0.42	0.074	13.5	2.4
55	Wet	Yes	171	0.53	0.206	17.1	6.6
70	Dry	No	Unknown	0.69	Unknown	22.1	Unknown
70	Dry	Yes	Unknown	0.60	Unknown	19.3	Unknown
70	Wet	No	Unknown	0.46	Unknown	14.7	Unknown
70	Wet	Yes	Unknown	0.37	Unknown	11.9	Unknown

Note: 1 mph = 1.61 km/h, 1 ft/s^2 = 0.3048 m/s^2



Note: 1 mph = 1.61 km/h, 1 ft/s^2 = 0.3048 m/s^2

Figure 2-1. Equivalent Constant Deceleration (a) for Nine Test Drivers – Based on Data in the NCHRP Report 400 (5).

Implicit in the selected deceleration threshold is the requirement that the vehicle braking system and pavement friction values can produce a deceleration rate of at least 11.2 ft/s^2 [3.4 m/s^2]. NCHRP Report 400 stated that skid data show that most wet pavement surfaces on state-maintained roadways exceed this threshold (5). The reported data were by functional class, and the initial speed used to generate the skid data was not provided. However, standard practice at that time was to use either 40 mph [65 km/h] and standard ribbed tire or 50 mph [80 km/h] and a smooth tire. In either case, the test speeds are much lower than the speeds being explored in this project. A recent TxDOT project (6) that explored pavement issues related to high-speed corridors stated:

> "Laser-based systems offer significant promise for estimating skid resistance at atypical high speeds. To apply TxDOT's current laser-based system to high-speed corridors, additional research will be required. Research should examine the suitability of some of the new pavement surfaces, adequacy of current test methods for measuring skid, resistance, physical properties of coarse aggregates, and related aggregate specifications."

In summary, the available deceleration rate information is very limited for higher speeds. Available pavement friction information does not reflect high operating speeds. Additional research in these areas is needed.

Driver Eye Height

The height of the driver's eye (assumed to be 3.5 ft [1.08 m]) should not vary due to a change in the design speed.

Object Height

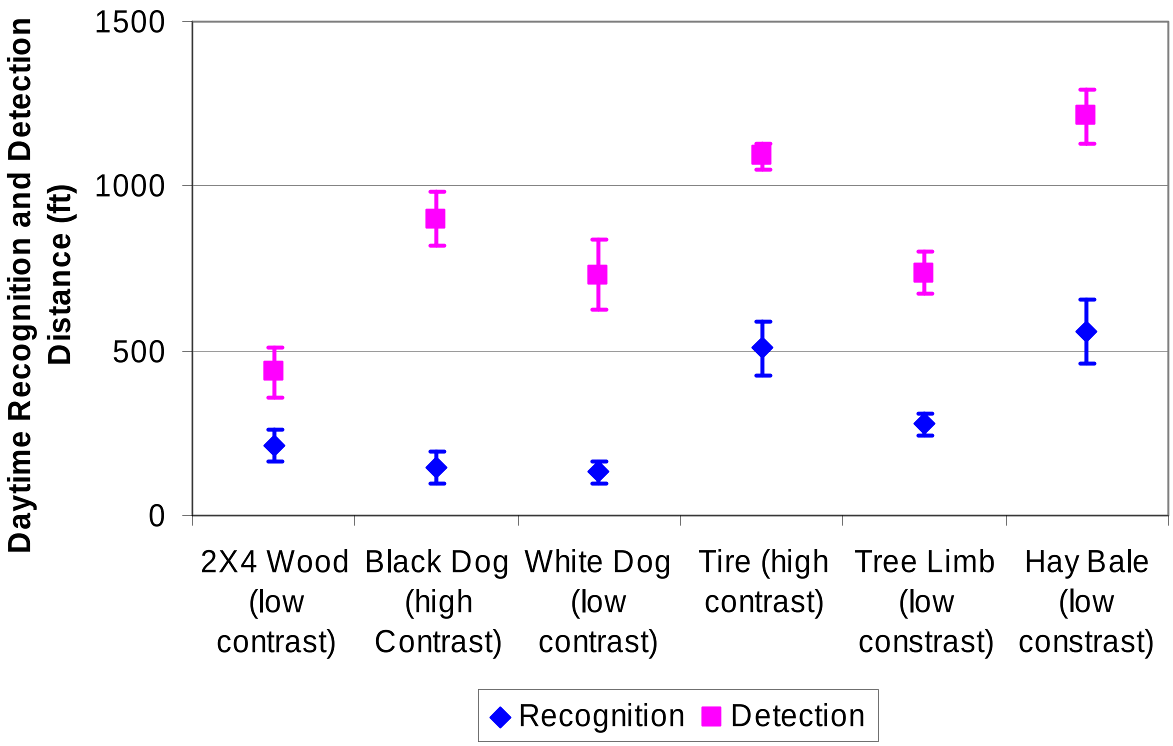
The height of the object is based on the height of a vehicle taillight and is assumed to be 2.0 ft [0.6 m]. It should not vary due to a change in the design speed.

Driver Visual Capability

"Seeing" an object requires both perception and recognition. Not only does a driver need to see the upper part of an object, but the driver needs to see enough of the object to be able to determine the level of hazard that object represents. Other factors affecting the detection of the object are luminance contrast, color contrast, ambient luminance level, and glare. At night, the perception and recognition of an object can also be influenced by headlamp visibility limits. Recognition is affected both by the appearance of an object and by driver expectations of what objects might be in a roadway.

NCHRP Report 400 reported on two studies that measured drivers' capabilities in detecting and recognizing different sized objects under different lighting conditions in a closed-course condition. Study 1 was conducted during daytime conditions with six objects ranging in height between 4 and 18 inches [102 and 457 mm] with varying contrasts. The 45 subjects represented a range of driver ages. Study 2 was conducted with 20 subjects – 10 drivers younger than 25 and 10 drivers 55 and older – and seven objects of different size and contrast during nighttime conditions. The initial speed for both studies was 55 mph [88 km/h]. Participants were instructed to indicate when they first detected an object and then to identify the object when it was recognizable. For Study 1, the participant was the driver. In Study 2, the participant was the front seat passenger (5).

The findings from the daytime visual capability studies indicated that drivers on level roadways can *detect* – but not *recognize* – a high contrast object at 495 ft [151 m] (the current stopping sight distance for a 55-mph [88 km/h] design speed). The NCHRP Report 400 authors noted that recognition is not totally necessary for stopping sight distance, but the driver must be able to identify the object as a hazard. Figure 2-2 shows the 95 percent confidence intervals for mean detection and recognition distances for the daytime study.



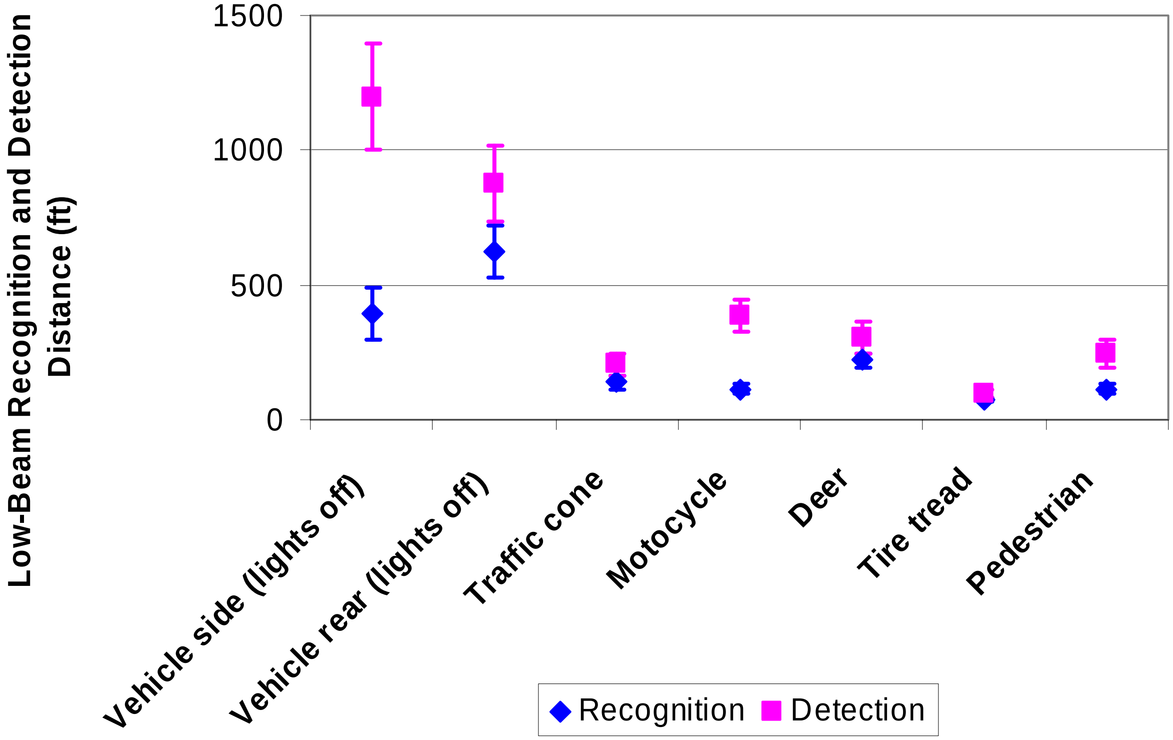
Note: 1 ft = 0.305 m

Figure 2-2. Daytime Detection and Recognition Distance with 95 Percent Confidence Intervals at 55 mph – Data from the NCHRP Report 400 (5).

The findings from the nighttime visual capability studies suggest that a substantial proportion of the driving population is not able to detect or recognize hazardous objects in the roadway at the then-current AASHTO minimum stopping sight distance for a 55-mph [88 km/h] design speed (430 ft [131 m]). The only exception to this statement is when the object is externally illuminated or retroreflective, that is, for the objects studied, had vehicle taillights or side reflectors. Detection, and more especially recognition of potentially hazardous objects at 430-ft [131 m] distances, is even more unlikely when low-beam headlights are in use (see Figure 2-3) (5).

The object currently used in the stopping sight distance procedure is the taillight of a vehicle (assumed to be 2.0 ft [0.6 m] in height). The nighttime study of 20 drivers found for low-beam conditions to an unlighted vehicle rear an average recognition distance of 625 ft [191 m] and detection distance of 875 ft [267 m] – both in excess of the current SSD distance for 55 mph [88 km/h] but not for higher design speeds. **A design speed of 80 mph [128 km/h] has a SSD of 910 ft [278 m], which exceeds the average detection distance of 875 ft [267 m]. Higher design speeds would also exceed the average detection distance.**

In summary, the limits of drivers' visual and cognitive capabilities are a concern for high design speeds. Additional investigation into drivers' capabilities would be desirable.



Note: 1 ft = 0.305 m

Figure 2-3. Low-Beam Detection and Recognition Distance with 95 Percent Confidence Intervals – Data from the NCHRP Report 400 (5).

Trucks

A recent TxDOT project (7) evaluated geometric design criteria currently used (in the early 2000s) to determine whether the criteria adequately reflected truck characteristics. Their recommendation for SSD applies to the situation when horizontal sight obstructions occur on downgrades, and particularly on long downgrades where truck speeds may exceed those of cars. The *Green Book* states that it is desirable to provide stopping sight distance greater than tabulated or computed values for design. The authors noted that the TxDOT *Roadway Design Manual* does not provide SSD corrections for grades (although it does refer designers to the *Green Book*), nor does it provide the caution noted above for designers regarding trucks on downgrades where horizontal sight obstructions can reduce the sight distance for truck drivers to equal that of passenger car drivers. They recommended adding a statement of caution regarding horizontal curves at the end of long downgrades to the *TRDM* for truck roadway design and that wording similar to that contained in the *Green Book* would be appropriate.

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

Based upon a review of the previous work that generated the recommendations for brake reaction time (2.5 s), the assumption should also be valid for speeds higher than 55 mph [88 km/h]. The review of the assumed deceleration rate (11.2 ft/s^2 [3.4 m/s^2] is not as clear. **The assumed deceleration rate may or may not be valid for speeds higher than 55 mph [88 km/h].** Test results using a 70-mph [112 km/h] initial speed were only available for three expert drivers, and their brake reaction time and deceleration rate were better than the current assumptions.

Stopping sight distances using the brake reaction time (2.5 s) and deceleration rate (11.2 ft/s^2 [3.4 m/s^2]) assumptions in the current equation are listed in Table 2-6 for design speeds of 85 to 100 mph [140 to 160 km/h] (see shaded area). The *Green Book* provides the methodology to calculate stopping sight distances for other grades. Table 2-7 lists the stopping sight distances on 3 percent grades.

While a driver may be able to stop a passenger vehicle in the distances listed in Table 2-6, it may not be possible for a driver to detect and recognize a hazard within those distances. The same concerns are present at an even greater level for nighttime conditions.

The issue of overdriving the distance illuminated by headlights is further exacerbated at higher speeds. This issue raises questions about considering provisions for continuous lighting and/or what the nighttime speed limit should be. Related to the nighttime speed limit is the question regarding the appropriate minimum speed limit for the facility.

Table 2-6. Potential Stopping Sight Distances for High Design Speeds (Using Existing Equation).

(US Customary)				
Design Speed (mph)	Brake Reaction Distance (ft)	Braking Distance on Level (ft)	Calculated Stopping Sight Distance (ft)	Design Stopping Sight Distance (ft)
85	312.4	693.5	1005.8	1010
90	330.8	777.5	1108.2	1110
95	349.1	866.2	1215.4	1220
100	367.5	959.8	1327.3	1330
(Metric)				
Design Speed (km/h)	Brake Reaction Distance (m)	Braking Distance on Level (m)	Calculated Stopping Sight Distance (m)	Design Stopping Sight Distance (m)
140	97.3	224.8	322.1	325
150	104.3	258.1	362.3	365
160	111.2	293.6	404.8	405
Shaded areas reflect high-design-speed potential values.				

Table 2-7. Potential Stopping Sight Distances for High Design Speeds on Grades (Using Existing Equation).

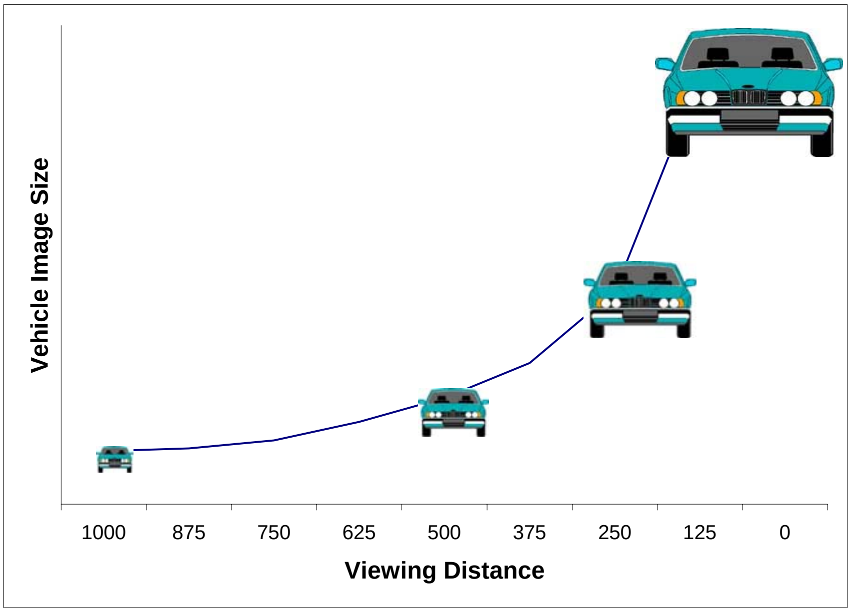
(US Customary)			(Metric)		
Design Speed (mph)	Stopping Sight Distance (ft)		Design Speed (km/h)	Stopping Sight Distance (m)	
	3% Downgrade	3% Upgrade		3% Downgrade	3% Upgrade
85	1070	950	140	343	304
90	1180	1045	150	387	342
95	1296	1145	160	433	381
100	1416	1250			
Shaded areas reflect high-design-speed potential values.					

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- Driver visual studies conducted at speeds higher than 55 mph [88 km/h] are needed. The studies need to be conducted both in daytime and nighttime conditions.
- Compare available friction for surface types proposed for use on high-design-speed roads to assumed deceleration rate.
- Brake reaction time and deceleration rate for drivers with initial speeds in the range expected on the high-design-speed roads are needed to verify that the 2.5 s and 11.2 ft/s^2 [3.4 m/s^2] assumptions are valid.
- Verify that the deceleration rate assumption is valid across a variety of vehicle types, especially those anticipated for a high-design-speed facility (e.g., sedans, sport utility vehicles [SUVs], recreational vehicles [RVs], heavy trucks, buses, etc.).

- Type of trucks expected on the facility along with their braking characteristics is needed.
- To date, research has focused on the detection of a fixed object. For high-design-speed roads, the concern of overtaking a slower moving vehicle is greater. Figure 2-4 shows a graphic illustrating the relationship between viewing distance and image size (*8*). The rate of change of an image (in terms of visual angle) is very small at far distances. The ability to perceive that a target is looming in a driver's visual field (that is, a driver is coming up behind a slow moving vehicle) depends on the driver's ability to detect that the image size is changing, which does not occur until the driver is fairly close (under 400 ft [122 m]), so at high speeds the driver won't have much time to decelerate or maneuver out of the way. On-road investigations have found that drivers' estimates of distance to an oncoming car are within 20 percent of the actual distance (*8*). Is the finding different when approaching the vehicle from the rear? In summary, what is the detection and recognition distance between a high speed vehicle and a slower moving vehicle? Is that distance sufficient to permit evasive maneuvers or stopping?
- With the potential need to improve visibility of objects during nighttime conditions, criteria for lighting of the facility are needed. The presence or absence of lighting could be a factor in establishing the nighttime speed limits or variable speed limits for the facility.
- Driver workload studies conducted at speeds greater than 55 mph [88 km/h] are needed. It is possible that driver workload would increase with higher speed, leading to an increased reaction time to hazards. In other words, at high speeds it may be that the driver is paying so much attention to the basic task of vehicle control that he or she may be slower in responding to hazards. The perception-reaction time values assumed by the AASHTO formulas may not be sufficient at higher speeds. On the other hand, driver vigilance may increase with higher speed, leading to equal or faster reaction times. It really is an open research question.



Note: 1 ft = 0.305 m

Figure 2-4. Relationship between Viewing Distance (ft) and Image Size (*8*).

CHAPTER 3

GRADE

The *Green Book* states that roadway grades "should be designed to encourage uniform operation throughout" (*2*). However, the *Green Book* follows this statement by adding that "few conclusions have been reached on the appropriate relationship of roadway grades to design speed." This statement is borne out by the lack of references in the *Green Book* specifically relating to roadway grades and a general lack of literature in this area. An exception has been in the critical area of truck performance on grades.

CURRENT GUIDANCE

The *Green Book* (*2*) and the TRDM (*1*) provide the recommended maximum grades for freeway-type facilities, and these recommendations are shown in Table 3-1 (both documents have the same values). Both metric and US customary units are shown to allow for comparison with international standards. AASHTO repeats this guidance in their standards for interstate highways (*9*). These recommendations are for mixed traffic (i.e., cars and trucks sharing the same lanes), and are assumed to principally affect trucks. Passenger cars are assumed to be able to climb a 4 or 5 percent grade under most circumstances without a significant speed loss (*2*). Table 3-1 does not provide any indication about maximum grade for design speeds above 80 mph [130 km/h].

Table 3-1. Recommended Maximum Grades for Urban/Suburban or Rural Freeways from TxDOT *Roadway Design Manual* (TRDM Table 2-9).

	Maximum Grade												
	(Metric)						(US Customary)						
	Design Speeds (km/h)						Design Speeds (mph)						
Terrain Type	**80**	**90**	**100**	**110**	**120**	**130**	**50**	**55**	**60**	**65**	**70**	**75**	**80**
Level (%)	4	4	3	3	3	3	4	4	3	3	3	3	3
Rolling (%)	5	5	4	4	4	4	5	5	4	4	4	4	4

OTHER GUIDANCE

A summary of international recommendations for maximum grades for divided highways and freeway facilities is shown in Table 3-2 (*10*). The US customary values are included for comparison purposes. The recommended maximum grades vary considerably, especially for design speeds above 81 mph [130 km/h]. For example, Italy chooses a 5 percent maximum grade even for design speeds up to 87 mph [140 km/h], perhaps reflecting the mountainous character of the country. Japan recommends a maximum grade of 2 percent at 75 mph [120 km/h], even though Japan is also a mountainous country. The United Kingdom sets a maximum grade for each roadway type regardless of terrain. The 3 percent grade shown in Table 3-2 is for a "dual carriageway" (i.e., divided highway) facility. Table 3-2 shows no international consensus about maximum grades, especially for design speeds above 81 mph [130 km/h].

Table 3-2. Recommended Maximum Grades, International (*10*).

Country	Design Speed, mph [km/h]						
	50 [80]	**56 [90]**	**62 [100]**	**68 [110]**	**75 [120]**	**81 [130]**	**87 [140]**
International							
Austria (%)	--	--	3	--	3	--	3
w/ climbing lane	--	--	6	--	--	5	4
France (%)	6	--	5	--	--	--	--
Germany (%)	6	5	4.5	--	--	--	--
Greece (%)	8	7	5	4.5	4	3	--
Switzerland (%)	8	--	6	--	4	--	--
Italy (%)	6	5	5	5	5	5	5
Canada (%)	4-6	4-5	3-5	3	3	3	--
Japan (%)	4	--	3	--	2	--	--
United Kingdom (%)	3	3	3	3	3	3	3
United States							
Level (%)	4	4	3	3	3	3	--
Rolling (%)	5	5	4	4	4	4	--
Mountainous (%)	6	6	5	5	--	--	--

Minimum design criteria and guidelines were developed for TxDOT for use on the Trans-Texas Corridor (TTC)-35 high-priority corridor (*11*). Their recommendations for grades are as follows:

Passenger Car Facility

- 3.0 percent (usual max) – vertical grades steeper than 3.5 percent shall be subject to approval by TxDOT, at its sole discretion;
- 4.0 percent (max); and
- 0.5 percent (min) – minimum grades shall not be less than 0.35 percent in toll plaza areas.

Truck Facility

- 2.0 percent (usual max) – vertical grades steeper than 2.5 percent shall be subject to approval by TxDOT, at its sole discretion;
- 3.0 percent (max); and
- 0.5 percent (min) – minimum grades shall not be less than 0.35 percent in toll plaza areas.

DISCUSSION

Vehicle Performance – Acceleration

An approach for investigating the relationship between grades and speed is to examine speed profiles. The 2004 *Green Book* provides speed profiles for a 200-lb/hp [120 kg/kw] truck ascending different sustained grades. Vehicle performance is the critical factor in determining maximum grade and critical length of grade, so this approach was explored. In addition, the use of a vehicle performance model would allow testing of a variety of different vehicles on a grade, including passenger cars, to determine how much these vehicles are affected by grades.

The *Green Book* defines the critical length of grade as the length of grade that would produce a speed reduction of 10 mph [16 km/h] for a 200-lb/hp [120 kg/kw] truck. The 10-mph [16 km/h] value (as opposed to a 15-mph [24 km/h] value) was included in the 1984 *Green Book* (*12*) which stated that the 10-mph [16 km/h] reduction "generally corresponds to the speed variation between adjacent levels of service."

TTI researchers consulted several references regarding vehicle performance *(13, 14, 15, 16)*. TTI generated a spreadsheet using assumptions of vehicle performance as documented in several sources that would easily permit comparisons between different assumptions. Table 3-3 lists the equations used in the spreadsheet. NCHRP Report 505 (*13*) includes discussion on truck characteristics with respect to critical length of grade. The authors developed a spreadsheet as part of their evaluation that can generate a truck speed profile on grade. The spreadsheet was included with their report. Because of the added effort to generate the multiple runs needed for the different grade comparisons. The NCHRP Report 505 spreadsheet was not the primary technique used to develop the recommendations for this project; rather, it was used as a quality control check.

The equations and assumptions used to reproduce the 2004 *Green Book* curves are summarized in Table 3-3. If Equation 3-1 is solved for a_v, a conclusion can be drawn about a vehicle's performance at a particular speed. For an uphill grade, if a_v is greater than zero, then that vehicle could increase its speed. Therefore, the vehicle is not affected by that uphill grade. If a_v is less than zero, that vehicle will decelerate to a lower speed. For a downhill grade, slightly different conditions prevail: if a_v is greater than zero, the vehicle requires braking to keep from accelerating; if a_v is less than zero, the vehicle will not roll down the grade on its own.

Equation 3-1 can be used to create speed profiles similar to those shown in Exhibit 3-55 in the 2004 *Green Book* (*2*). This exhibit, along with NCHRP Report 505 results, was used to verify the accuracy of the vehicle performance model. The final crawl speed estimates matched well (on the order of less than 2 mph [3 km/h]).

The speed-distance curves for a typical heavy truck of 200 lb/hp [120 kg/kw] for deceleration on upgrades for an 85-mph [137 km/h] initial speed were created and are shown in Figure 3-1. **Note that according to this figure a 200-lb/hp [120 kg/kw] truck can not sustain an 85-mph [137 km/h] operating speed even on a level grade. After 1 mile [1.61 km] on the level grade the truck is traveling at less than 80 mph [128 km/h] and in less than 3 miles [5 km] the truck is at 75 mph [120 km/h], which is 10 mph [16 km/h] slower than the 85-mph [137 km/h] design speed.** The TTI spreadsheet was used to determine which initial speeds result in trucks decelerating on a level grade. Initial speeds of 70 and 75 mph [112 and 120 km/h] result in a truck accelerating or maintaining the initial speed on the level grade. For initial speeds of 80 mph [128 km/h] and higher, trucks slow on level grades. Using the NCHRP Report 505 spreadsheet produces even more restrictive findings than those found with the TTI spreadsheet. The 200-lb/hp [120 kg/kw] truck does not maintain speed on a level grade when the initial speed is as low as 70 mph [112 km/h] (slows to 68 mph [110 km/h] in about 1.5 miles [2.4 km]). For initial speeds of 75 mph [120 km/h] and 80 mph [128 km/h], the truck slows to 70 mph [112 km/h] and 71 mph [115 km/h], respectively, in approximately 1.5 miles [2.4 km]. Research is needed to verify these findings as the anticipated operating speed of trucks on high-design-speed roadways can have a major impact on the performance of the system.

Table 3-3. Equations to Predict Speeds of Vehicles on Grade.

The force required to move a vehicle on a grade at a given speed is reflected by the following equation:

$$F_v = \frac{W_v}{g} a_v = \frac{P_a}{V} - F_r - F_a - F_g \tag{3-1}$$

Where:

- F_v = force required to move vehicle at speed V, lb;
- W_v = weight of the vehicle, lb;
- g = acceleration of gravity, 32.2 ft/s^2;
- a_v = acceleration of the vehicle, ft/s^2;
- P_a = power available at the drive wheels, lb·ft/s;
- V = vehicle speed, ft/s;
- F_r = rolling resistance, lb;
- F_a = aerodynamic drag, lb; and
- F_g = grade effect, lb.

The power available at the drive wheels is a function of the engine horsepower. It is estimated as:

$$P_a = 0.85 P_e \tag{3-2}$$

Where:

- P_e = engine hp, lb·ft/s.

The rolling resistance F_r is the resistance of the vehicles' tires, bearings, and transmission to forward movement. For a truck with radial tires, the rolling resistance can be approximated by:

$$F_r = 0.001 \times W_v \times (C_1 + C_2 V) \tag{3-3}$$

Where:

- C_1 = initial coefficient, assumed to be 6 for radial tires, 5.3 for mixed tires; and
- C_2 = second coefficient, assumed to be 0.068 for radial tires, 0.044 for mixed tires.

F_r increases linearly as speed increases. The coefficients shown in Equation 3-3 were selected to mimic the results shown in Exhibits 3-55 in the 2004 *Green Book* for a starting speed of 70 mph.

The grade effect F_g is the conversion of kinetic to potential energy as the vehicle ascends the grade.

$$F_g = W_v \sin\alpha \tag{3-4}$$

Where:

- α = angle of the grade, tan^{-1}(grade/100).

Aerodynamic resistance F_a is drag on the vehicle as it moves through the atmosphere.

$$F_a = 0.5 p_a C_D A_f V^2 \tag{3-5}$$

Where:

- p_a = air density, 0.0023 lb·s^2/ft^4;
- C_D = drag coefficient; and
- A_f = frontal area of the vehicle, ft^2.

For trucks, NCHRP Report 505 recommends a drag coefficient C_D of 0.6 for trucks with aerodynamic aids and 0.7 for a truck without them (*13*). C_D is between 0.3 and 0.35 for passenger cars, 0.35 and 0.41 for SUVs, and 0.4 to 0.45 for pickup trucks, depending on the exact model. The frontal area *A* of a truck with a van trailer is approximately 114 ft^2, based on the design truck dimensions in NCHRP Report 505 (*13*). The frontal area of passenger cars may vary considerably, depending on the vehicle.

The distance traveled by a vehicle can be determined from:

$$s = Vt + 0.5 a_v t^2 \tag{3-6}$$

Where:

- s = distance traveled, ft; and
- t = time increment (assumed as 1 s in the evaluation), s.

Research has shown that even when drivers are not limited by vehicle performance, driver's preferred acceleration rate is limited as a function of the magnitude of the difference between the driver's current speed and the desired speed. The following three equations represent limitations on new speeds based on maximum preferred acceleration for drivers for three specific cases (*13*):

Case I: $$If \left|V_d - V\right| \le 1.2, then \rightarrow V_n = V_d \tag{3-7}$$

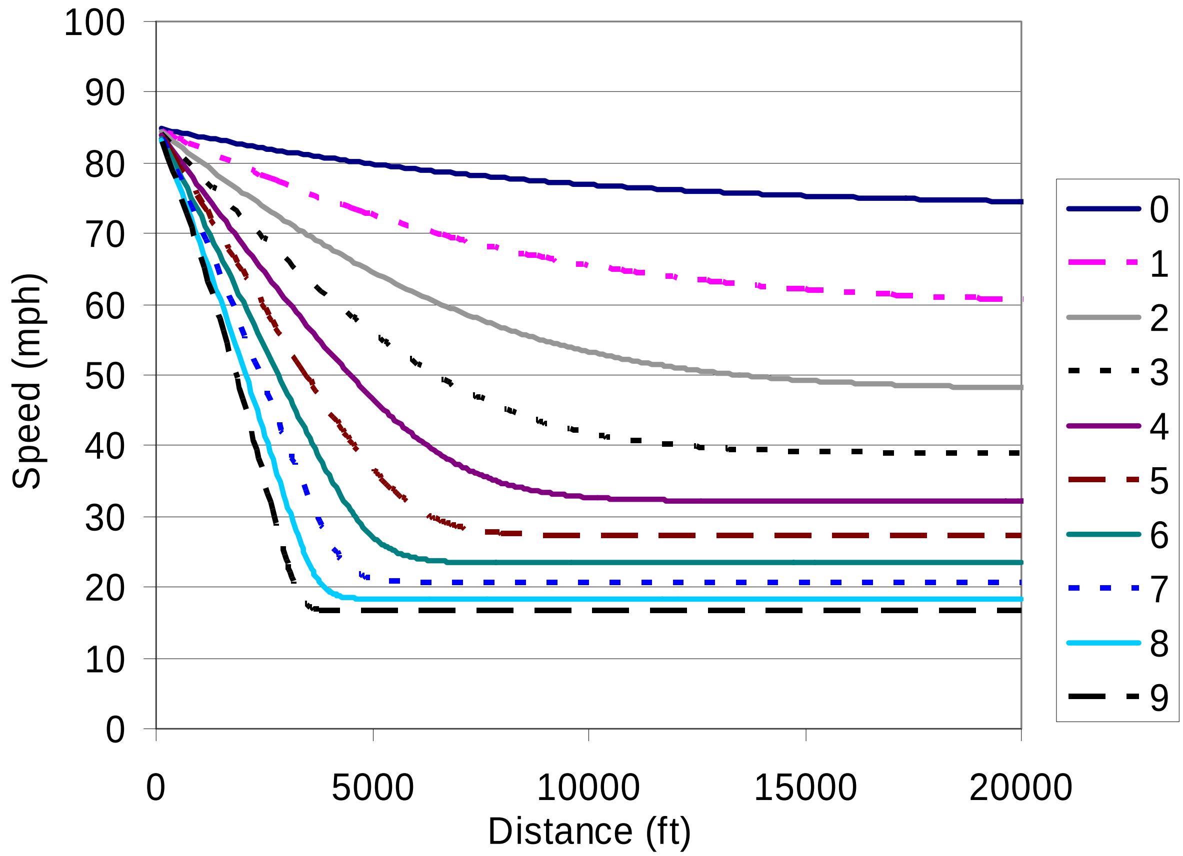
Case II: $$\begin{gathered} If \left|V_d - V\right| > 1.2 and V_d - V > 0 \\ then \rightarrow V_n = V + (1.2 + 0.108\left|V_d - V\right|)t \end{gathered} \tag{3-8}$$

Case III: $$\begin{gathered} If \left|V_d - V\right| > 1.2 and V_d - V < 0 \\ then \rightarrow V_n = V - 1.2t \end{gathered} \tag{3-9}$$

Where:

- V_d = driver designed speed, unit?;
- V = vehicle speed (ft/s) at start of time interval t; and
- V_n = new speed (ft/s) at the end of time interval t.

Note: 1 ft = 0.305 m, 1 mph = 1.61 km/h, 1 ft/s^2 = 0.3048 m/s^2



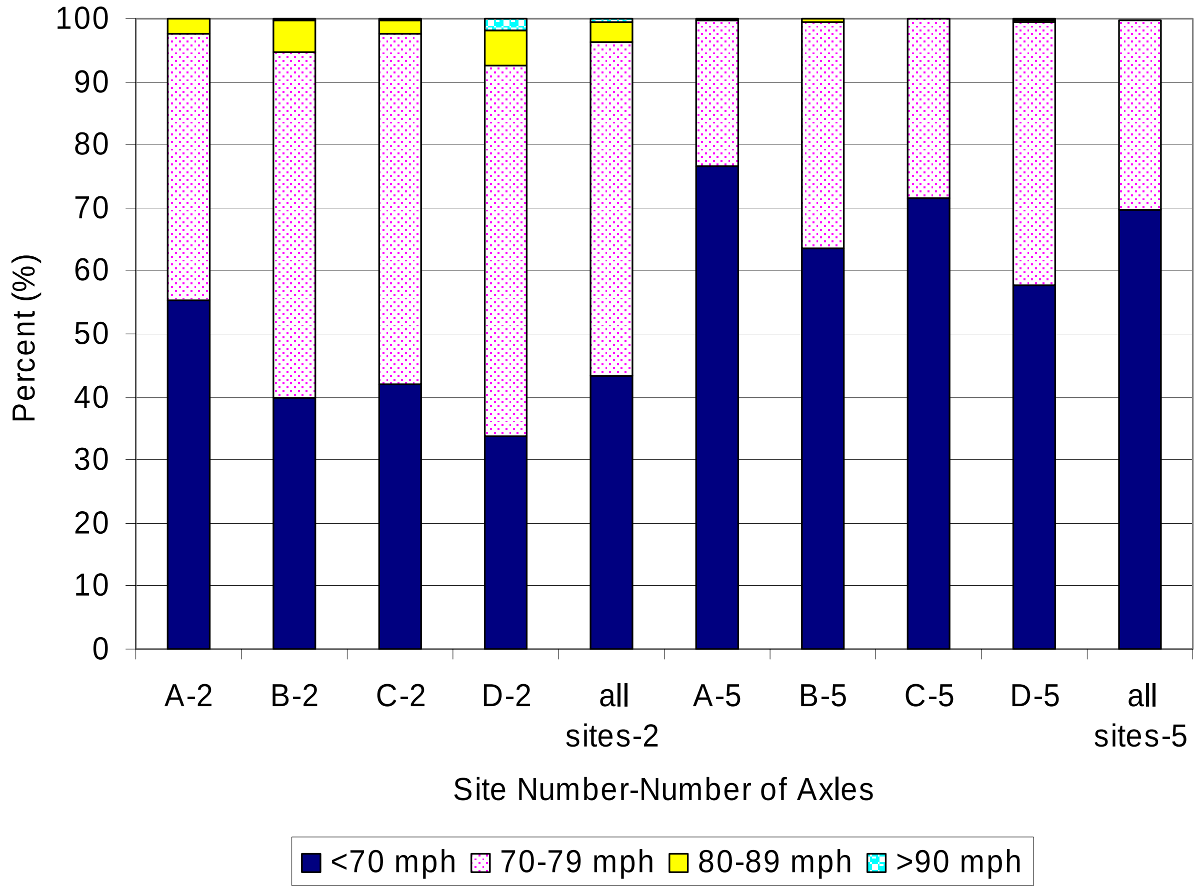
Note: 1 ft = 0.305 m, 1 mph = 1.61 km/h

Figure 3-1. Speed-Distance Curves for a Typical Heavy Truck of 200 lb/hp [120 kg/kw] for Deceleration on Upgrades of 0 to 9 Percent, 85-mph [137 km/h] Design Speed.

Sample of Texas Speed Data

A sample of speed data was obtained for four Texas rural highway locations to identify if trucks currently perform at speeds in the range being considered in this document. Trucks can be redesigned to operate efficiently at higher speeds if or when the legal speed limit is raised. The goal of this effort was to see what percentage of vehicles currently operate in the speed ranges that the equations predicted could not be maintained by a 200-lb/hp [120 kg/kw] truck. The data set included speeds for all vehicles for a one-week period at the four sites. Focusing on two- and five-axle vehicles, the percent of vehicles in 10-mph [16 km/h] speed bins was determined. Figure 3-2 shows the distribution. As expected, the five-axle vehicles did operate at lower speeds as compared to two-axle vehicles (stated in another manner, a larger proportion of five-axle vehicles were in compliance with a 70-mph [112 km/h] speed limit than two-axle vehicles). Approximately 4 percent of the two-axle vehicles operate at speeds in excess of 80 mph [128 km/h]. For the five-axle trucks, less than 1 percent operate at over 80 mph [128 km/h]. This data set represented almost 300 five-axle vehicles. In summary, existing data show that a sample of large trucks operate at speeds that exceed the limiting values produced by the equations.

Therefore, additional research could refine the equations to produce better speed predictions or could clarify the type of speed being predicted (e.g., average, 85th percentile, maximum, etc.).



Note: 1 mph = 1.61 km/h

Figure 3-2. Distribution of Speeds for a Sample of Two- and Five-Axle Vehicles at Four Texas Rural Highways.

Critical Length of Grade

The *Green Book* defines the critical length of grade as the length of grade that would produce a speed reduction of 10 mph [16 km/h] for a 200-lb/hp [120 kg/kw] truck *(2)*. The 200-lb/hp [120 kg/kw] truck is intended to represent typical conditions in the United States. NCHRP Report 505 conducted field studies to determine truck weight-to-power ratios ranges for freeways and two-lane highways *(13)*. The available data suggest that truck performance is better for the freeway truck population than for the two-lane highway truck population and the truck population in western states is better than in eastern states. The authors of NCHRP Report 505 found the 85th percentile truck weight-to-power ratios to range from 170 to 210 lb/hp [102 to 126 kg/kw] for the truck population on freeways. Specifically, they found 183 lb/hp [111 kg/kw] on a California freeway, 169 lb/hp [103 kg/kw] on a Colorado freeway, and 207 lb/hp [126 kg/kw] on a Pennsylvania freeway. Therefore, continued use of the 200-lb/hp [120 kg/kw] value for this evaluation would represent a conservative assumption, although a check will be made to determine if the recommendations would change if a 169-lb/hp [103 kg/kw] ratio is assumed.

Experimentation with the vehicle performance model indicated that passenger vehicles are not significantly affected by grades as steep as 3 percent, regardless of initial speed, unless the passenger car in question has a high weight-to-power ratio. Therefore, the selection of grades can be performed with trucks only, just as it is for lower speeds.

Vehicle Performance – Deceleration

Consideration should be given to the expected speed differentials that could occur on high- speed roadways due to grades along with normal variation in traffic speeds and vehicle types. A driver of a vehicle traveling 85 mph [137 km/h] may not be able to adequately judge his or her rate of gain on a slower vehicle in sufficient time to safely adjust for the situation (by changing lanes, slowing down, etc.). This issue is also discussed in the Research Needs subsection of Chapter 2. The speed differential may also change with increasing operating speeds. Further research should shed light on the effect of speed differential on safety for high-speed roadways.

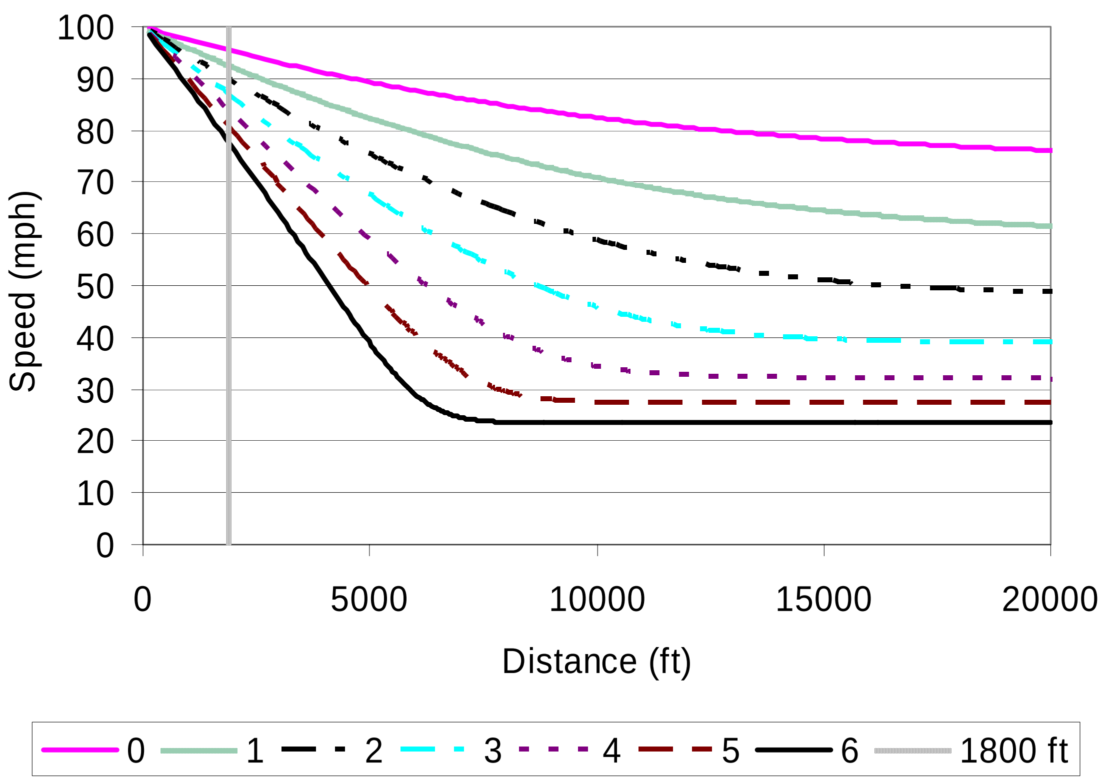
Calculations

To use the speed-distance curves to identify a recommended maximum grade for a design speed, the distance traveled before reaching the acceptable speed reduction of 10 mph [16 km/h] is needed. This distance could be an assumed value. However, better guidance may be provided by determining what the existing recommended grade and initial speed combinations would represent. Plots were generated for initial speeds of 50, 55, 60, 65, 70, and 80 mph [80, 88, 96, 105, 112, and 128 km/h]. With each plot, the critical length of grade was measured for the recommended grade and a 10-mph [16 km/h] speed reduction. For example, a 200-lb/hp [120 kg/kw] truck's speed would be 10 mph [16 km/h] less at what distance from the start of the climb? For a 3 percent grade and an initial speed of 65 mph [105 km/h] the answer was approximately 1900 ft [580 m]. The critical lengths of grade estimated for initial speeds between 50 and 80 mph [80 and 128 km/h] are shown in Table 3-4. The critical lengths of grade ranged between 1600 and 2400 ft [488 and 732 m] with a rounded average of 1850 ft [564 m] using the TTI spreadsheet. Using the NCHRP Report 505 spreadsheet generated slightly shorter critical lengths of grade – an average of 1575 ft [480 m] rather than 1850 ft [564 m] (see Table 3-4).

The TTI spreadsheet was then used to generate similar curves for assumed initial speeds of 85, 90, 95, and 100 mph [137, 145, 153, and 161 km/h]. The curves were then inspected to identify which grade curve would produce the nearest (but not over) 10-mph [16 km/h] speed reduction at an approximate 1850-ft [564 m] critical length of grade. Figure 3-3 shows an example for the 100-mph [161 km/h] design speed. To improve readability, a portion of the plot is enlarged and shown in Figure 3-4. The curve with a 10-mph [16 km/h] speed drop closest to an 1850-ft [564 m] critical length of grade is the 2 percent curve. Therefore, for a 100-mph [161 km/h] design speed, the suggested maximum grade for trucks is 2 percent. The suggestions for the other grades are listed in Table 3-5. The recommendations for different weight/hp trucks are also listed in Table 3-5. The recommendations are similar for three types of trucks except for the more efficient truck (169 lb/hp [103 kg/kw]) on the lowest design speed investigated – for 85-mph [137 km/h] design speed the more efficient truck would result in a 3 percent grade being the maximum grade, while the lesser efficient trucks result in a 2 percent grade being the maximum grade.

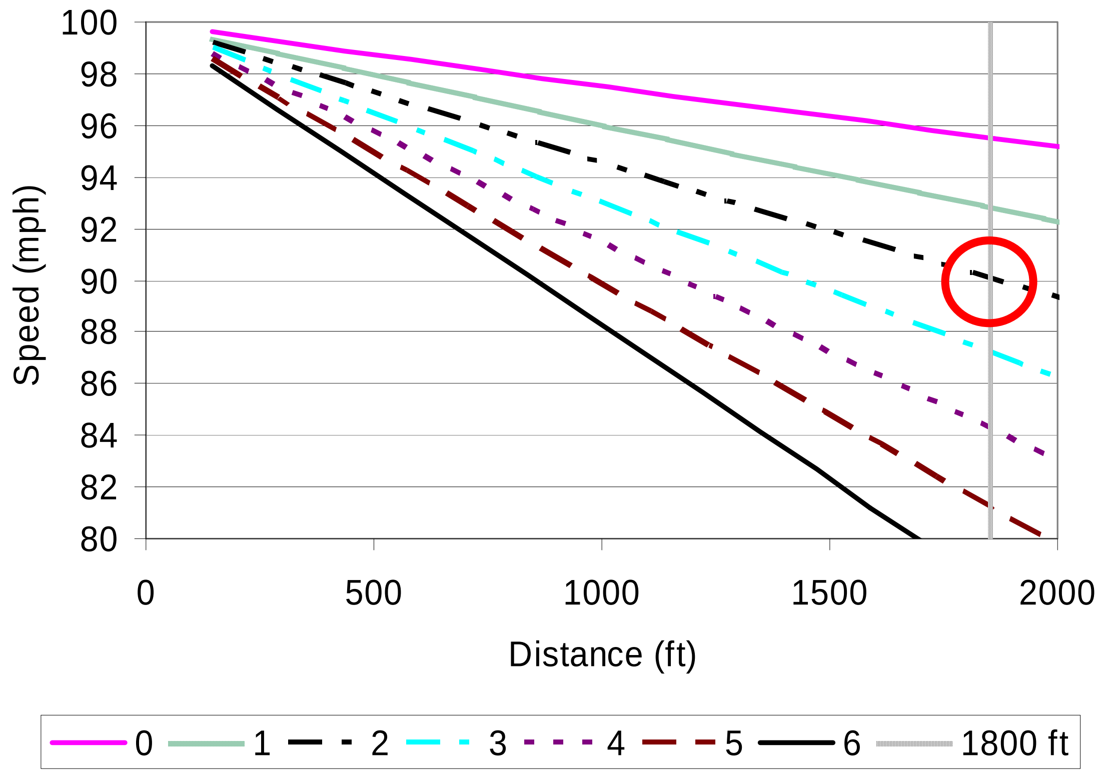
Table 3-4. Critical Length of Grade Estimates.

Initial Speed (mph)	Recommended Grade for Level Terrain (%)	Critical Length of Grade as Measured by 10 mph Speed Reduction (ft)	
		Using TTI Spreadsheet	Using NCHRP Report 505 Spreadsheet
50	4	1800	1500
55	4	1600	1400
60	3	2400	1900
65	3	1900	1700
70	3	1900	1600
75	3	1750	1500
80	3	1650	1400
Rounded average critical length of grade		**1850**	**1575**
Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m			



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 3-3. Speed-Distance Curves for a Typical Heavy Truck of 200-lb/hp [130 kg/kw] for Deceleration on Upgrades of 0 to 6 Percent, 100-mph [161 km/h] Design Speed.



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 3-4. Speed-Distance Curves for a Typical Heavy Truck of 200-lb/hp [120 kg/kw] for Deceleration on Upgrades of 0 to 6 Percent, 100-mph [161 km/h] Design Speed, Close-up View.

Table 3-5. Suggested Grades Using Different Truck Types and Spreadsheets.

Initial Speed (mph)	Recommended Grade (%) for Level Terrain			
	200 lb/hp Truck Table 3-3 Equations[a]	169 lb/hp Truck Table 3-3 Equations[a]	220 lb/hp Truck Table 3-3 Equations[a]	200 lb/hp Truck NCHRP Report 505 Spreadsheet[b]
85	2	3	2	2
90	2	2	2	2
95	2	2	2	2
100	2	2	2	2

[a] Critical length of grade assumed to be 1850 ft based on data in Table 3-4.
[b] Critical length of grade assumed to be 1575 ft based on data in Table 3-4.
Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

The potential maximum grades are listed in Table 3-6.

Table 3-6. Potential Maximum Grades.

Terrain Type	Existing (mph)							Potential (mph)			
	50	55	60	65	70	75	80	85	90	95	100
Level (%)	4	4	3	3	3	3	3	2	2	2	2
Rolling (%)	5	5	4	4	4	4	4	--	--	--	--
Shaded areas reflect high-design-speed potential values.											
Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m											

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- The spreadsheet developed to predict truck speeds as part of this project along with the spreadsheet developed as part of NCHRP Report 505 indicate that trucks cannot maintain the design speeds (85 to 100 mph [137 to 161 km/h]) investigated in this project on level grades and on upgrades for segments as short as 1 mile [1.6 km]. Review of existing data at a sample of Texas sites did show that several trucks were operating in the 85- to 100-mph [137 to 161 km/h] range. However, those vehicles represented only a very small percentage of the trucks at the sites. Research is needed to update or expand the truck speed equations to better predict the speeds for these vehicles. The anticipated operating speed of trucks on the high-design-speed roadways could have a major impact on the performance of the system.
- The speed effects on high-speed vehicles of a continuous downgrade may be worth additional investigation to identify cautions on locating horizontal curvature following downgrades of specific lengths.
- An area of necessary research involves the effects of speed differential on highway safety at high speeds. Additional discussion on this issue is in Chapter 2. How grades affect the development of speed differential (say between a grade-influenced heavy vehicle and a passenger car) needs to be part of the research.
- Stopped vehicles are less of a concern in freeway environments except when they are accelerating away from a rest area, ramp, or weight station. Chapter 14 discusses the length needed for acceleration to expected operating speed for a ramp.

CHAPTER 4

VERTICAL CURVES

Vertical curves create a gradual transition between different grades. The proper design of these transitions is essential for the safe and efficient operation of a roadway. This chapter discusses crest and sag vertical curves.

CURRENT GUIDANCE

This section outlines the current guidance used in the United States to design crest and sag vertical curves, as outlined in the *Green Book* (*2*). TxDOT's *Roadway Design Manual* (*1*) uses the same procedures. The lengths of both types of curves are controlled by the available sight distance.

Crest Vertical Curves

The *Green Book's* procedure uses a parabolic curve to connect the sections of tangent grade. The design of crest vertical curves involves using the stopping sight distance S and the difference between the two grades A to find the length of the vertical curve L (*2*).

If the sight distance S is less than the length of the crest vertical curve L, then:

US Customary	Metric	
$L = \frac{AS^2}{2158}$	$L = \frac{AS^2}{658}$	**(4-1)**

If the sight distance S is greater than the length of the crest vertical curve L, then:

US Customary	Metric	
$L = 2S - \frac{2158}{A}$	$L = 2S - \frac{658}{A}$	**(4-2)**

Equations 4-1 and 4-2 assume a driver eye height of 3.5 ft [1.08 m] and an object height of 2.0 ft [0.6 m], in accordance with the *Green Book*'s design recommendations. By setting L equal to S and solving for A in either Equation 4-1 or 4-2, the value of A where the length of the vertical curve equals the required sight distance can be found. Then, dividing S by A provides the design values of K shown in *Green Book* Exhibit 3-71. The *Green Book* calls for a minimum crest vertical curve length of three times the design speed of the roadway (*2*).

Sag Vertical Curves

Unlike crest curves, sag vertical curves do not typically have sight distance restrictions unless an overpass structure is present. Instead, the main criterion is headlight sight distance (HSD).

As with crest curves, the design of sag vertical curves involves using the stopping sight distance *S* and the difference between the two grades *A* to find the length of the vertical curve *L*.

If the sight distance *S* is less than the length of the sag vertical curve *L*, then:

US Customary	**Metric**	
$L = \frac{AS^2}{400 + 3.5S}$	$L = \frac{AS^2}{120 + 3.5S}$	**(4-3)**

If the sight distance *S* is greater than the length of the crest vertical curve *L*, then:

US Customary	**Metric**	
$L = 2S - \frac{400 + 3.5S}{A}$	$L = 2S - \frac{120 + 3.5S}{A}$	**(4-4)**

Equations 4-3 and 4-4 assume a headlight height of 2.0 ft [0.6 m] and an upward light spread of 1 degree from horizontal in accordance with the *Green Book*'s design recommendations. By setting *L* equal to *S* and solving for *A* in either Equation 4-3 or 4-4, the value of *A* can be found for a length of the vertical curve equal to the required sight distance. Then, dividing *S* by *A* provides the design values of *K* shown in *Green Book* Exhibit 3-74. Also as with crest curves, the minimum recommended length for sag curves is three times the roadway design speed *(2)*.

In addition, the *Green Book* notes that the design curve lengths for headlight sight distance are considerably more than the curve lengths that are necessary for driver comfort. The guidelines in the *Green Book* cite a maximum centripetal acceleration of 1 ft/s^2 [0.3 m/s^2] for driver comfort. The following equation (*Green Book* Equation 3-53) is provided to calculate the resulting vertical curve length *(2)*:

US Customary	**Metric**	
$L = \frac{AV^2}{46.5}$	$L = \frac{AV^2}{395}$	**(4-5)**

As an alternative to using Equation 4-5, the 2004 *Green Book* makes the following statement:

> "The length of vertical curve needed to satisfy this comfort factor at the various design speeds is only about 50 percent of that needed to satisfy the headlight sight distance criterion for the normal range of design conditions" *(2)*.

TRDM contains a similar statement. However, the statement above is no longer accurate due to the changes made to the *Green Book's* stopping sight distance calculations as a result of the

NCHRP SSD project (*5*). The reason for this difference will be discussed fully in the discussion section of this chapter.

OTHER GUIDANCE

International

Lamm et al. (*10*) used the *Green Book* procedure to illustrate its use in international practice. The mechanics are the same for all applications. However, different countries use different assumptions about driver reaction times and deceleration rates that result in different required stopping sight distances and thus vertical curvature criteria.

TTC-35

Minimum design criteria and guidelines were developed for TxDOT for use on the TTC-35 high priority corridor (*11*). Their recommendations for minimum *K*-values are:

<u>Passenger Car Facility</u>

- Crest – 384
- Sag – 231

<u>Truck Facility</u>

- Crest – 384
- Sag – 231

Literature

Some literature is available on vertical curves. However, much of the available literature deals with rural two-lane highways, which may be considerably different from the freeway environment of the Trans-Texas Corridor. As an example, Fambro et al. (*17*) found that available sight distance appears to influence mean speed reduction between the upstream segment and the crest vertical curve. For two-lane rural highway sites without shoulders and with a crest vertical curve inferred design speed of less than 40 mph [65 km/h], a mean reduction in speed of about 3.6 mph [5.8 km/h] was found. For crest vertical curve inferred design speeds between 40 and 49 mph [65 and 79 km/h], the mean speed reductions were on the order of 1.8 to 3.0 mph [2.9 to 4.8 km/h]. Another finding from the study was that the operating speeds at all of the crest vertical curves used in the study exceeded the inferred design speed. Fitzpatrick et al. (*18*) found that limited stopping sight distance on vertical curves was not necessarily a safety hazard if there were no other hazards present. Both of these studies were for two-lane highways and may not apply to freeways.

Hassan and Easa (*19, 20, 21*) have argued for many years, separately and together, for changes to AASHTO and Canadian vertical curve design guidelines. Similarly, Thomas et al. (*22*) point out that the *Green Book* does not account for braking on the crest curve and propose a procedure to adjust vertical curve design to account for braking on the curve. The *Green Book*'s sight distance requirements have been modified since these papers were presented, and it is not clear if any of these suggestions would be an improvement. Taiganidis *(23)* proposes using a "crash speed" to determine the severity of a crash related to a crest vertical curve.

There is relatively little literature about sag vertical curves. Sag curves typically do not have sight distance issues unless an overpass is present, and the *Green Book*'s comfort guidelines are usually not the limiting values in design. Hassan *(21)* includes sag curves in his proposed modifications. Thomas et al. *(22)* also include sag curves in their presentation but present no design modifications for sag curves.

DISCUSSION

Drainage

Very long vertical curves are known to have long sections of roadway that are essentially level. Under normal circumstances, the normal roadway cross slope is sufficient to drain these sections of pavement. However, if superelevation is being introduced near the flattest part of the curve, the superelevation transition may create a section of roadway with either a very small cross slope or no cross slope at all. These sections are vulnerable to ponding during rain, which can result in driver loss of control if the water depth is sufficient. All vertical curves that would be used for high-speed roadways would have *K* values exceeding the drainage limit of 167, so additional attention would need to be paid to these curves during the design process to ensure adequate drainage.

Sag Vertical Curve Lengths for Driver Comfort

As mentioned earlier, the *Green Book* states that the length of sag vertical curves for driver comfort is typically about half of the length of sag vertical curves for headlight sight distance *(2)*. The history of this statement requires a look at previous editions of highway design policies in the United States.

The design of sag vertical curves typically uses the simple relationship outlined in the "Current Guidance" section. Simply put, the length of a vertical curve is equal to the rate of elevation change per percent change in grade multiplied by the percent change in grade, or:

$$L = KA \qquad \textbf{(4-6)}$$

For sag vertical curves designed for comfort, the equations would look like:

$$L\text{-}comfort = K\text{-}comfort \times A \qquad \textbf{(4-7)}$$

With:

US Customary / **Metric**

$$K\text{-}comfort = \frac{V^2}{46.5} \qquad K\text{-}comfort = \frac{V^2}{395} \qquad \textbf{(4-8)}$$

Because *A* is the same for both Equations 4-6 and 4-7, it cancels out of the calculations. Therefore, the ratio of the length of a sag vertical curve for comfort, *L-comfort*, to the length of a sag vertical curve for headlight sight distance, *L*, is the same as the proportion of the *K* values for those lengths, and can be written as:

US Customary

$$\frac{L\text{-}comfort}{L} = \frac{AV^2/46.5}{KA} = \frac{V^2/46.5}{K} = \frac{K\text{-}comfort}{K} \qquad \textbf{(4-9)}$$

Metric

$$\frac{L\text{-}comfort}{L} = \frac{AV^2/395}{KA} = \frac{V^2/395}{K} = \frac{K\text{-}comfort}{K}$$

Equation 4-9 was used to calculate the final column of Tables 4-1, 4-2, and 4-3.

Equation 4-5 first appears in the design of vertical curves contained in the 1965 *A Policy on Geometric Design of Rural Highways*, commonly known as the *Blue Book* *(24)*. The *K* values for headlight sight distance used in the *Blue Book* and the *K-comfort* values for driver comfort are shown in Table 4-1. It is important to note that the *Blue Book* used running speed for determining headlight sight distance but used the design speed for determining driver comfort. Therefore, the ratio of sag curve length for comfort and sag curve length for sight distance is approximately 0.75, as shown in the final column of Table 4-1. The *Blue Book* also contains the following statement about the ratio of curve lengths:

> "The length of vertical curve needed to satisfy this comfort factor at the various design speeds is only about *75 percent* of that needed to satisfy the headlight sight distance criterion for the normal range of design conditions" *(24)*. [Emphasis added]

This is the same statement as the current *Green Book*, except that the proportion reflects the use of running speed for headlight sight distance and design speed for driver comfort, as shown in Table 4-1.

Table 4-1. *K* Values and Ratios for Sag Vertical Curves Using 1965 *Blue Book* Design. Values (*24*).

Design Speed (mph)	Minimum stopping distance[a] (ft)	Design *K-HSD*[a]	Calculated *K-comfort*[b]	Ratio *K-comfort* to *K-HSD*[c]
30	200	35	19	0.54
40	275	55	34	0.62
50	350	75	54	0.72
60	475	105	77	0.73
65	550	130	91	0.70
70	600	145	105	0.72
75	675	160	121	0.76
80	725	185	138	0.75

[a] Minimum stopping distance and *K-headlight sight distance* were calculated using the running speed (rather than design speed) according to the *Blue Book*.
[b] Driver comfort *K* values are calculated using the design speed, as stated in the *Blue Book*.
[c] The ratio used the *K-comfort* determined using design speed, divided by the *K-HSD* determined using running speed.
Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

The next change in design values for sag vertical curves occurred in the 1984 *Green Book* (*12*). The change involved the use of a range of stopping sight distances instead of only designing for the stopping sight distance associated with running speed. This change created a range of design *K-HSD* values for vertical curves. However, the lengths of sag vertical curves for driver comfort were still to be calculated using design speed (1984 *Green Book* page 312). The values of *K* from the 1984 *Green Book* and the resulting changes to the ratio of length for driver comfort to the length for headlight sight distance are shown in Table 4-2. At this time, the statement about the relationship between the length for driver comfort and for headlight sight distance was changed from 75 percent to its current value of 50 percent. The last two columns of Table 4-2 list the ratio calculated using various combinations of *K-comfort* and *K-HSD*. The only combination that generated ratios greater than about 50 percent was when *K-comfort* was calculated using design speed and *K-HSD* was calculated using running speed (which was the technique used in the 1965 *Blue Book*). Why these higher ratios were not mentioned in the 1984 *Green Book* (*12*) is not clear. The 50 percent ratio can reflect the situation when the speed used to calculate *K-comfort* matched the speed used to calculate *K-HSD* (see final column in Table 4-2) or when design speed is used to calculate *K-HSD* and design speed is used to calculate *K-comfort*.

Table 4-2. *K* Values and Ratios for Sag Vertical Curves Using 1984 *Green Book* (*12*) Design Values.

Design {Running} Speed (mph)	Stopping Sight Distance Range[a] (ft)	Design *K-HSD* Range[a]	Calculated *K-comfort* Range[b]	Ratio of *K-comfort* (design speed) to *K-HSD* (running-design speed)[c]	Ratio of *K-comfort* (running-design speed) to *K-HSD* (running-design speed)[d]
20 {20}	125-125	20-20	9-9	0.46-0.46	0.46-0.46
25 {24}	150-150	30-30	12-13	0.55-0.55	0.51-0.55
30 {28}	200-200	40-40	17-19	0.53-0.53	0.46-0.53
35 {32}	225-250	50-50	22-26	0.62-0.54	0.52-0.54
40 {36}	275-325	60-70	28-34	0.62-0.50	0.50-0.50
45 {40}	325-400	70-90	34-44	0.63-0.49	0.50-0.49
50 {44}	400-475	90-110	42-54	0.60-0.49	0.47-0.49
55 {48}	450-550	100-130	50-65	0.63-0.50	0.48-0.50
60 {52}	525-650	120-160	58-77	0.63-0.49	0.47-0.49
65 {55}	550-725	130-180	65-91	0.70-0.52	0.50-0.52
70 {58}	625-850	150-220	72-105	0.70-0.49	0.48-0.49

[a] Stopping sight distance and headlight sight distance are based on both the running speed and design speed, as shown in the 1984 *Green Book*.
[b] Per the 1984 *Green Book*, *K-comfort* values are to be calculated using design speed. Initial value is based on running speed, second value is based on design speed.
[c] The first ratio in the column used *K-comfort* determined using design speed divided by *K-HSD* determined using running speed. The second value used *K-comfort* determined using design speed divided by *K-HSD* determined using design speed. Restricting *K-comfort* to only using design speed mirrors the technique implied in the 1965 *Blue Book*.
[d] The first ratio in the column used *K-comfort* determined using running speed divided by *K-HSD* determined using running speed. The second value used *K-comfort* determined using design speed divided by *K-HSD* determined using design speed. This approach matches *K-HSD* with *K-comfort* determined using similar types of speed (either design or running).

Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

The same ratio values were used in the 1990 edition of the *Green Book* (*25*), and the metric version was used in the 1994 *Green Book* (*26*).

In 2001, the equation for stopping sight distance was changed from using a pavement friction factor to using a constant deceleration, which reduced the stopping sight distances by as much as 15 percent. Also, running speed was no longer used to calculate stopping sight distance. Table 4-3 shows the results of these changes. Note that the *K-comfort* values based on design speed have not changed compared to Table 4-1 or Table 4-2. However, the stopping sight distances have changed (an increase from the 1965 values in Table 4-1 and a decrease from the 1984 values in Table 4-2), which in turn changes the design *K* values for headlight sight distance and the resulting ratios.

Table 4-3. *K* Values and Ratios for Sag Vertical Curves Using 2001 or 2004 *Green Book* Design Values.

(US Customary)				
Design Speed (mph)	**Stopping Sight Distance (ft)**[a]	**Design *K-HSD***[a]	**Calculated *K-Comfort***[b]	**Ratio of *K-Comfort* to *K-HSD***
15	80	10	5	0.50
20	115	17	9	0.53
25	155	26	13	0.50
30	200	37	19	0.51
35	250	49	26	0.53
40	305	64	34	0.53
45	360	79	44	0.56
50	425	96	54	0.56
55	495	115	65	0.57
60	570	136	77	0.57
65	645	157	91	0.58
70	730	181	105	0.58
75	820	206	121	0.59
80	910	231	138	0.60
85	1010	260	155	0.60
90	1110	288	174	0.60
95	1220	319	194	0.61
100	1330	350	215	0.61
(Metric)				
Design Speed (km/h)	**Stopping Sight Distance (m)**[a]	**Design *K-HSD***[a]	**Calculated *K-Comfort***[b]	**Ratio of *K-Comfort* to *K-HSD***
20	20	3	1	0.34
30	35	6	2	0.38
40	50	9	4	0.45
50	65	13	6	0.49
60	85	18	9	0.51
70	105	23	12	0.54
80	130	30	16	0.54
90	160	38	21	0.54
100	185	45	25	0.56
110	220	55	31	0.56
120	250	63	36	0.58
130	285	73	43	0.59
140	325	84	50	0.59
150	365	96	57	0.59
160	405	107	65	0.61

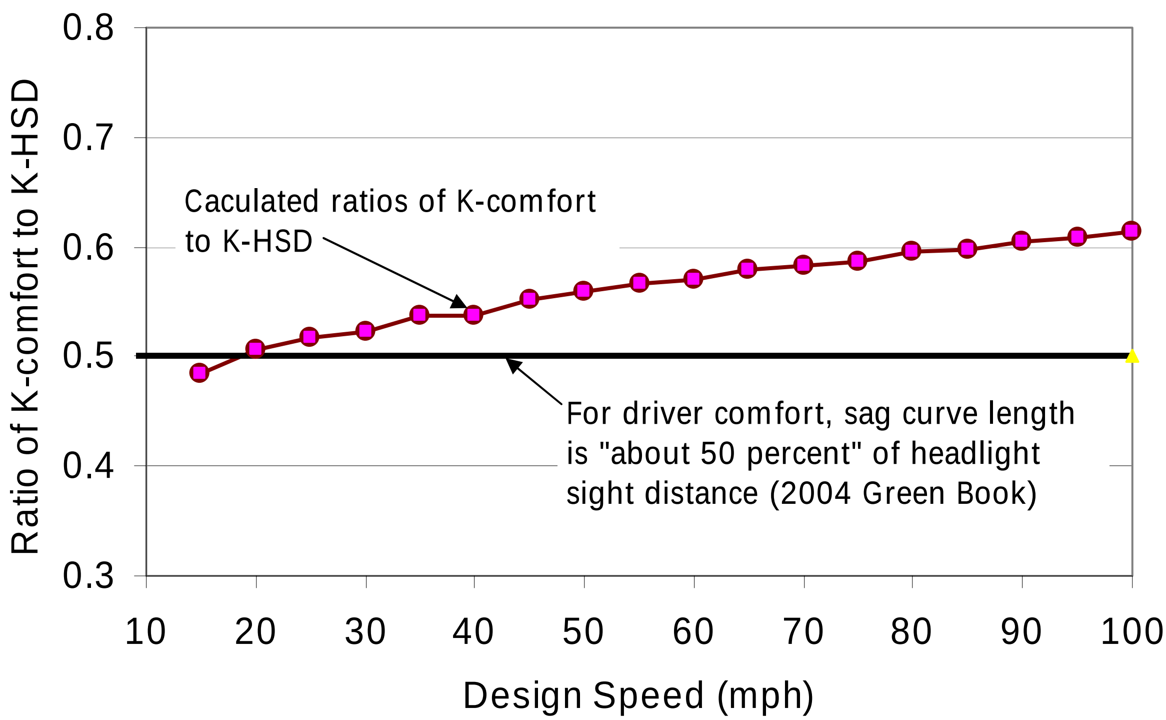
[a] Stopping sight distance and *K*-headlight sight distance were calculated using only the design speed, as shown in the 2004 *Green Book*.

[b] *K-comfort* values are calculated using design speed.

Shaded areas reflect high-design-speed potential values.

The results of these changes are shown in the final column of Table 4-3 and graphically in Figure 4-1. For very low speeds, the length of a sag vertical curve for driver comfort is approximately 50 percent of the length of a sag vertical curve for headlight sight distance. However, for "high-speed" roadways (i.e., design speeds of 50 mph [80 km/h] or more), sag vertical curve length for driver comfort is now between 55 percent and 60 percent of the sag vertical curve length for headlight sight distance. The statement that the length of a sag vertical curve for driver comfort is approximately 50 percent of the length of a sag vertical curve for headlight sight distance is no longer accurate for all speeds and especially not for high speeds.

Table 4-3 and Figure 4-1 also show the associated *K* ratios for design speeds of 85, 90, 95, and 100 mph [137, 145, 153, and 161 km/h]. The length of a sag vertical curve for driver comfort is approximately 60 percent of the length of the same vertical curve for headlight sight distance, rather than the 50 percent stated in the *Green Book*.



Note: 1 mph = 1.61 km/h

Figure 4-1. *K* Ratios for Driver Comfort on Sag Vertical Curves versus Headlight Sight Distance.

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

In the absence of clear evidence to the contrary, the *Green Book*'s procedures for crest and sag vertical curves appear to be reasonable for design speeds higher than 80 mph [130 km/h]. The *K* values summarized in Table 4-4 would be the results of exercising the *Green Book*'s equations for crest and sag vertical curves, respectively, for design speeds higher than 80 mph [130 km/h].

The ratio of the length of a sag vertical curve that satisfies the driver comfort criteria to the length of a sag vertical curve for headlight sight distance is shown in Table 4-3 for all design speeds. For design speeds above 80 mph [130 km/h], a sag vertical curve satisfying the driver comfort criterion will be about 60 percent of the length of a sag vertical curve designed for headlight sight distance.

Table 4-4. Potential Design Controls for Crest and Sag Vertical Curves for High Design Speeds.

(US Customary)					
Design Speed (mph)	**Assumed Stopping Sight Distance (ft)**	**Crest Vertical Curves**		**Sag Vertical Curves**	
		Calculated *K*	**Design *K***	**Calculated *K***	**Design *K***
85	1010	472.7	473	259.2	260
90	1110	570.9	571	287.5	288
95	1220	689.7	690	318.7	319
100	1330	819.7	820	349.9	350
(Metric)					
Design Speed (km/h)	**Assumed Stopping Sight Distance (m)**	**Crest Vertical Curves**		**Sag Vertical Curves**	
		Calculated *K*	**Design *K***	**Calculated *K***	**Design *K***
140	325	160.5	161	84.0	84
150	365	202.5	203	95.3	96
160	405	249.3	250	106.7	107
Shaded areas reflect high-design-speed potential values.					

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- The effects of minimum length vertical curves on driver safety, comfort, and performance at high design speeds are not known at this time and may require closer investigation.
- Additional guidance regarding drainage needs on long vertical curves could be of value.

CHAPTER 5

LANE WIDTH

CURRENT GUIDANCE

The TxDOT *Roadway Design Manual* (*1*) states in Chapter 2, Section 6, Cross Sectional Elements:

> "For high-speed facilities such as all freeways and most rural arterials, lane widths should be 12 ft [3.6 m] minimum."

Chapter 3, Section 6, Freeways states:

> "The minimum and usual mainlane width is 12 ft [3.6 m]."

The *Green Book* states in the freeway chapter that freeways should have a minimum of two through-traffic lanes for each direction of travel and those lanes should be 12 ft [3.6 m] wide (*2*).

OTHER GUIDANCE

Minimum design criteria and guidelines were developed for TxDOT for use on the TTC-35 high-priority corridor by Cintra (*11*). Their recommendations for lane width were 12 ft [3.6 m] for a passenger car facility and 13 ft [4 m] for a truck facility.

Lamm et al. (*10*) recommends that the basic lane width be obtained by adding the width of the lateral moving space to the width of the design vehicle. The lateral moving space is defined as the space needed by a non-track-guided vehicle to compensate for driving and steering uncertainties as well as for safety distances for lateral projecting parts (like mirrors) or lateral loading overhangs. For roads in cross section group A (which would include Texas freeways) Lamm et al. recommend a width of 4.1 ft [1.25 m] for the lateral moving space and 8.2 ft [2.5 m] for the design vehicle resulting in a 12.3 ft [3.75 m] basic lane width (or 11.5 ft [3.50 m] for "exceptional cases").

DISCUSSION

Safety Relationships

Previous research shows a definite relationship between lane width and safety on rural two-lane highways. See, for example, material developed for the draft prototype chapter of the *Highway Safety Manual* (*27*). A 2005 TxDOT report documented the safety relationship between lane width and crashes on rural two- and four-lane highways in Texas (*28*). Lane widths less than 12 ft [3.6 m] had a higher number of crashes predicted using the regression equations developed as part of the research. There were insufficient data for the 13-ft [4 m] lane width to identify whether 13 ft [4 m] would be predicted to have fewer crashes than 12 ft [3.6 m]. A 2004 paper in *Accident Analysis and Prevention* examined the Highway Safety Information System data for the state of Illinois *(29)*. The data presented were not sufficient to examine the effects for each road category (i.e., functional class). Therefore, the findings include the full range of functional classes (freeways to urban local collectors). The findings indicate that increased lane width has no statistically significant effect on total crashes. However, it appears to be associated with increased fatalities. Unfortunately, the numerical ranges used in the analysis of lane width are not provided. Also, due to the study methodology, it is not known if this relationship holds true for freeways. An analysis of Oregon crashes *(30)* found a counterintuitive effect between lane width and crashes for freeways – crashes increased with increased lane widths on urban freeways. In their literature review the authors noted that one previous study found a similar relationship between lane width and crash frequency, but two studies found lower crash frequencies on road segments with wider lanes. The authors noted that the risk homeostasis theory (behavior adapts to changes in perceived hazards) may explain that drivers are changing their behavior due to perceptions of reduced risks for wider travel lane segments.

Trucks

The lane width criteria in the *Green Book* were established without reference to any explicit vehicle width specification. NCHRP Report 505 *(13)* did note that research has shown a definite relationship between lane width and safety on two-lane roads. That research has not indicated if the observed effect relates directly to truck widths, however.

The Surface Transportation Assistance Act of 1982 mandated that states allow 8.5-ft [2.6 m] vehicle widths on a national network. All but one of the design trucks included in the 2004 *Green Book* includes an overall width dimension of 8.5 ft [2.6 m]. Passenger cars have the narrowest design width for the design vehicles included in the 2004 *Green Book* with a value of 7 ft [2.1 m]. Most vehicles have widths of 8 to 8.5 ft [2.4 to 2.6 m] (see Exhibit 2-1 of the *Green Book*). The one exception is the farm tractor vehicle with a width of 8 to 10 ft [2.4 to 3.1 m]. NCHRP Report 505 reviewed truck characteristics as factors in roadway design. The report made several recommendations on updating the design vehicles dimensions. However, none of the recommendations were for the vehicle width dimension.

Rearward amplification is the amplification of the magnitude of steering corrections in the rear trailers of multi-trailer truck combinations. NCHRP Report 505 notes that there is no indication that rearward amplification of sufficient magnitude to require lane widths greater than 11 to 12 ft [3.3 to 3.6 m] occurs with sufficient frequency that wider lanes are needed.

Mason et al. *(31)* in a 1986 TxDOT study on exclusive truck facilities recommended the following formula for lane widths where trucks are adjacent to existing travel lanes:

$$W = W_v + 4.5 \text{ ft} \qquad \textbf{(5-1)}$$

Where:

W = width of lane, ft; and
W_v = width of the vehicle, ft.

Using the 8.5-ft [2.6 m] design truck width produces a 13-ft [4 m] lane width for trucks.

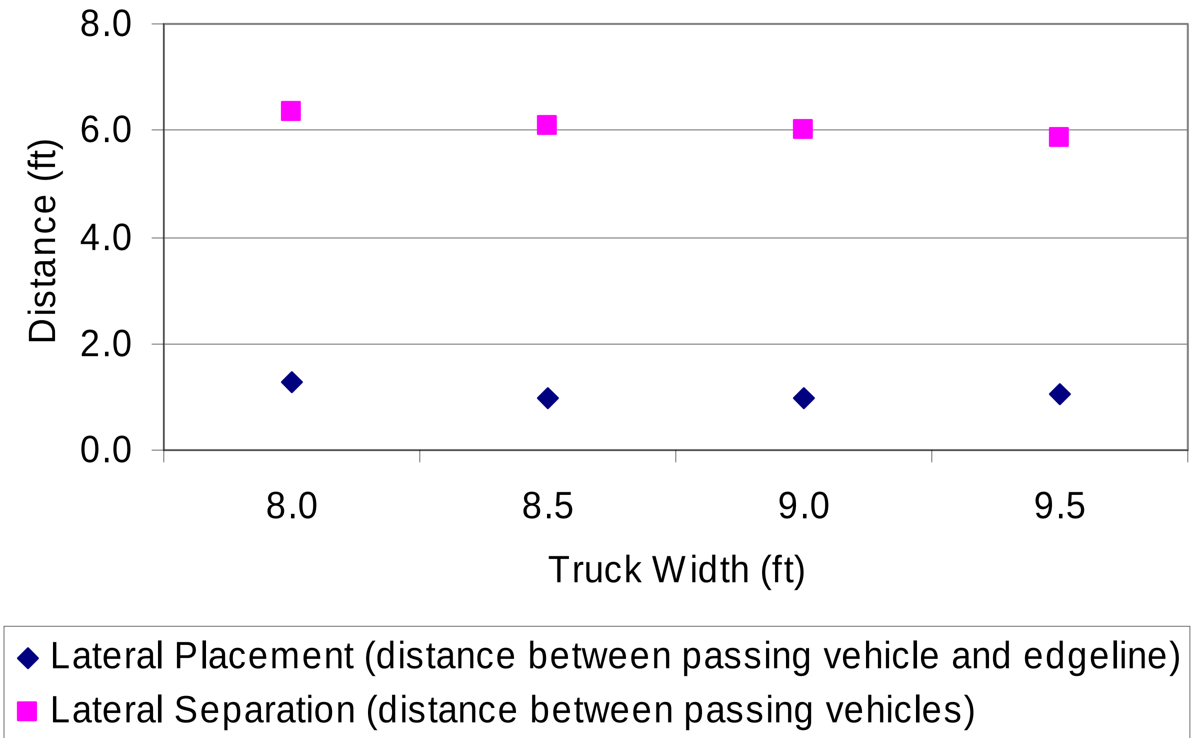
A 2003 TxDOT project *(7)* reviewed design criteria with respect to trucks to identify recommended changes to the *TxDOT Roadway Design Manual*. They noted that for mixed-flow lanes, the 8.5-ft [2.6 m] vehicles still have ample width on 12-ft [3.6 m] lanes, but consideration should be given to the probability of the roadway becoming an exclusive truck roadway. Their final recommendations were:

- using minimum lane width of 12 ft [3.6 m] for high-speed facilities such as freeways,
- increasing lane width from 12 ft [3.6 m] to 13 ft [4 m] for exclusive truck facilities, and
- keeping 12-ft [3.6 m] lanes where trucks remain in a mixed flow or are restricted to specific lanes within a facility.

Lane Positioning

Two older studies were identified that have addressed the operational effects of wider vehicles and the implications of these effects for highway design. A joint National Highway Traffic Safety Administration (NHTSA)-FHWA assessment *(32)* conducted in 1973 compared the operational effects of 8- and 8.5-ft [2.4 and 2.6 m] wide buses on two-lane, four-lane, six-lane, and eight-lane highways based on research reported in the literature. The research found that cars shift their lateral position by 12 to 18 inches [300 to 460 mm] when a bus was present, but the magnitude of this shift did not vary between 8.0- and 8.5-ft [2.4 and 2.6 m] wide buses.

A 1982 FHWA study *(33)* of the effects of truck width on the positions of passing vehicles was conducted on two-lane highways. Vehicle widths of 8.0, 8.5, 9.0, and 9.5 ft [2.4, 2.6, 2.7, and 2.9 m] were created by changing the width of a fabricated wood and aluminum box on the trailer. As truck width increased, less room was available between the truck and far edge of road, and the distance between the truck and passing vehicle decreased. The authors reported that drivers of the passing vehicles adjusted for wider trucks by moving away from the truck and reducing their distance to the road edge. Therefore, increasing truck width was found to lead to a reduction in both lateral separation and lateral placement. Figure 5-1 illustrates the findings for lateral placement (distance between passing vehicle and edgeline) and lateral separation (distance between passing vehicles) from the study. Overall, when passing a 9.5-ft [2.9 m] truck as compared to passing a 8.0-ft [2.4 m] truck, vehicles were about 4 inches [101 mm] closer to the edgeline and about 6 inches [152 mm] closer to the truck they were passing. The study concluded that there was no effect of truck widths on shoulder encroachments in the passing maneuvers, which were observed consistently in about 6 percent of the passes. The authors commented that drivers would attempt to find a compromise between moving away from the larger vehicle and the risk of leaving the lane or roadway.



Note: 1 ft = 0.305 m

Figure 5-1. Vehicle Placement When Passing a Truck on a Two-Lane Rural Highway (*33*).

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

Information on the variability in lane positioning due to neighboring vehicles and due to high speeds is limited. Relationships between freeway lane width and speed or crashes are also not well established. Therefore, clear direction on values for appropriate lane widths at high speeds is not present. However, logical lane width values represent only a small range (say 11 to 14 ft [3.4 to 4.3 m]). Potential lane width values for two types of facilities are listed in Table 5-1.

Table 5-1. Potential Lane Width Values for High-Design-Speed Roads.

Passenger Car/Mixed-Use Facility	Truck Facility or Mixed-Use with High Truck Volumes[a] Facility
12 ft [3.6 m]	13 ft [4.0 m]

[a] High truck volumes occur when the directional design hourly volume (DDHV) for truck traffic exceeds 250 veh/h.
Shaded areas reflect high-design-speed potential values.

With the possibility of having an exclusive truck facility in addition to a passenger car or mixed-use facility, controlling criteria values may vary between the two types of facilities. A question is when should the different set of values be used? Previous research (*7*) has stated that a wider outside shoulder is to be used when the average daily truck traffic is at least 5000 trucks per day during the design period. The *Green Book* indicates to use wider shoulders when the directional design hourly volume for truck traffic exceeds 250 veh/h. While these values are for using a wider *shoulder* they could also be logical for deciding on when to use a wider *lane*. A recent TxDOT research project *(34)* developed a more extensive procedure (rather than only relying on truck volume) for determining when to use different truck treatments. The research developed a *Truck Facility Guidebook* to provide criteria to assist in choosing among three types of truck facilities: 1) lane restrictions, 2) dedicated truck lanes, and 3) exclusive truck roadways. Because the *Guidebook* did not include when to consider wider lanes (or shoulders) and to provide consistency for when a wider shoulder is to be considered, Table 5-1 includes the note that a wider lane is applicable for mixed-use facilities when the truck volume (250 veh/h) identified in the *Green Book* is present.

If a single lane width for high speeds is preferred, then a lane width of 13 ft [4 m] is suggested.

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- Identify the relationship between lane width and safety for rural freeways. Is there a relationship between lane width and speed for rural freeways? Will wider lanes encourage higher speeds? Are wider lanes associated with lower crash rates?
- Validate and refine the constants used by other researchers in determining lane widths (i.e., Mason's 4.5 constant and Lamm's 4.1 constant). Should these constants be different for a high-speed roadway? Is there more drift and steering corrections being made at the higher speeds? Is there an increased shy distance from neighboring large vehicles? Does the shy distance vary depending upon whether the vehicle is in an outer lane (and can move closer or onto the shoulder) or is in the center lane?
- Should the outer lanes have a wider width to assist with facilitating recoveries or should the additional pavement be allocated to the shoulders?
- Identify the needed lane width on horizontal curves for the high-speed roadways.
- Previous research has stated that a wider outside shoulder is to be used when the average daily truck traffic is at least 5000 trucks per day during the design period. The *Green Book* indicates to use a wider shoulder when the directional design hourly volume for truck traffic exceeds 250 veh/h. Should exceeding a specific truck volume result in the use of a wider lane? If so, which of these values is (or what is the value) for when a wider lane width is appropriate for a high-design-speed roadway? Should other conditions (such as total daily traffic) be considered in the decision-making process? How does the design speed affect the decision-making process?
- A recent TxDOT project developed a *Truck Facility Guidebook* to provide criteria to assist TxDOT in choosing among three types of truck facilities: 1) lane restrictions, 2) dedicated truck lanes, and 3) exclusive truck roadways *(34)*. Additional research is needed to determine if (or how) the recommendations from that effort would change with the higher design speeds. The new research effort could also identify criteria for if (or when) a wider lane should be used for a mixed-use facility.
- Drivers normally demonstrate some variability in their lateral lane position, even on a tangent section, due to slight steering corrections as they adjust to forces from the road surface, vehicle wheel alignment, and wind. Normally, these steering adjustments are

performed subconsciously and result in the vehicle maintaining a straight path. Occasionally, drivers may drift from their marked lane due to momentary inattention or perhaps environmental effects such as a strong wind gust and quickly correct their steering to re-enter their lane. On higher speed roads, the effects of steering adjustments are magnified greatly. The actual vehicle dynamics involved in the relationship between steering wheel position and vehicle path are quite complicated. They involve the steering wheel gain, the vehicle suspension, size and condition of tires, pavement surface characteristics, and speed of the vehicle. A given amount of steering input will result in larger vehicle path deflections at higher speeds than at lower speeds. A literature search found no research on driver steering behavior at high speeds beyond trained racing drivers. All of the past research on steering behavior has generally dealt with emergency maneuvers. The safety and human factors concern is that people inexperienced in driving at high speeds will use their habitual steering responses that have served them well at traditional highway speeds. These responses will result in much greater vehicle path changes at the higher speeds that may surprise the drivers, causing them to over-correct in the opposite direction. At high speeds, this type of steering behavior could lead to overturning the vehicle very quickly. There is even less known about how heavy vehicles' steering responds at these higher speeds. Like with passenger cars, a truck driver's normal steering inputs could have unexpected consequences at high speeds, leading to jackknifing and overturning. More thorough vehicle dynamic analysis is possible with existing modeling tools, but is beyond the scope of the current project. These concerns have implications for design guidelines concerning lane width, shoulder width, and longitudinal rumble stripe presence and offset.

CHAPTER 6

SHOULDER WIDTH

CURRENT GUIDANCE

The TxDOT *Roadway Design Manual* states in Chapter 2, Section 6, Cross Sectional Elements:

> "Wide, surfaced shoulders provide a suitable, all-weather area for stopped vehicles to be clear of the travel lanes. Shoulders are of considerable value on high-speed facilities such as freeways and rural highways. Shoulders, in addition to serving as emergency parking areas, lend lateral support to travel lane pavement structure, provide a maneuvering area, increase sight distance of horizontal curves, and give drivers a sense of safe, open roadway" (*1*).

The TxDOT *Roadway Design Manual* states in Chapter 3, Section 6, Freeways:

> "Continuous surfaced shoulders are provided on each side of the mainlane roadways, both rural and urban…. The minimum widths should be 10 ft [3.0 m] on the outside and 4 ft [1.2 m] on the median side of the pavement for four-lane freeways. On freeways of six lanes or more, 10 ft [3.0 m] inside shoulders for emergency parking should be provided. A 10 ft [3.0 m] outside shoulder should be maintained along all speed change lanes with a 6 ft [1.8 m] shoulder considered in those instances where light weaving movements take place. See Table {6-1} for future information" (*1*).

Table 6-1. Roadway Widths for Controlled Access Facilities (US Customary) from TxDOT *Roadway Design Manual* (TRDM Table 3-18).

Type of Roadway	Inside Shoulder Width (ft)	Outside Shoulder Width[b] (ft)	Traffic Lanes (ft)
Mainlanes–4-lane divided	4	10	24
Mainlanes–6-lane or more divided	10	10	36[a]
1-Lane direct connect[b]	2 (roadway) 4 (street)	8	14
2-Lane direct connect	2 (roadway) 4 (street)	8	24
Ramps[b] (uncurbed)	2 (roadway) 4 (street)	6 (min), 8 (des)	14
Ramps[c] (curbed)	--	--	22

[a] For more than six lanes, add 12 ft width per lane.
[b] If sight distance restrictions are present due to horizontal curvature, the shoulder width on the inside of the curve may be increased to 8 ft and the shoulder width on the outside of the curve decreased to 2 ft (roadway) or 4 ft (street).
[c] The curb for a ramp lane will be mountable and limited to 4 inches or less in height. The width of the curbed ramp lane is measured face to face of curb. Existing curb ramp lane widths of 19 ft may be retained.

Note: 1 ft = 0.305 m

The *Green Book* states in the Freeway Chapter that paved shoulders should be continuous on both the right and left sides of all freeway facilities. The usable paved width of the right shoulder should be at least 10 ft [3.0 m] where the DDHV for truck traffic exceeds 250 veh/h, the right shoulder width should be 12 ft [3.6 m]. On four-lane freeways, the median (or left) shoulder is normally 4 to 8 ft [1.2 to 2.4 m] wide, at least 4 ft [1.2 m] of which should be paved and the remainder stabilized. On freeways of six or more lanes, the usable paved width of the median shoulder should also be 10 ft [3.0 m] and preferably 12 ft [3.6 m] where the DDHV for truck traffic exceeds 250 veh/h (*2*).

OTHER GUIDANCE

Minimum design criteria and guidelines were developed for TxDOT for use on the TTC-35 high priority corridor (*11*). Their recommendations for inside shoulder width were 6 ft [1.8 m] (interim), 10 ft [3.1 m] (ultimate) for a passenger car facility, and 6.5 ft [2.0 m] (interim) and 10 ft [3.1 m] (ultimate) for a truck facility. For outside shoulder width, their recommendations were 10 ft [3.1 m] for both passenger car and truck facilities with a note that if future widening is to the outside shoulder then the outside shoulder width shall be 12 ft [3.6 m] for passenger car facility or 13 ft [4.0 m] for truck facility.

DISCUSSION

Safety Relationships

Previous research has shown a definite relationship between shoulder width and safety on two-lane highways. See, for example, material developed for the draft prototype chapter of the *Highway Safety Manual* (*27*). A 2005 TxDOT report documented the safety relationship between crashes and lane and shoulder widths on rural two- and four-lane highways in Texas (*28*). While the data show a range of number of crashes for a given shoulder width, overall, the trend is smaller number of crashes for wider shoulders. Prediction equations were developed and can be used to predict number of crashes for a given lane and shoulder width, average daily traffic (ADT), and length of segment. A 2004 paper in *Accident Analysis and Prevention* examined the Highway Safety Information System data for the state of Illinois (*29*). The data were not sufficient to examine the effects for each road category (i.e., functional class). Therefore, the findings are for the full range of functional classes (freeways to urban local collectors). The findings indicate that "increases in outside shoulder width appear to be associated with a decrease in accidents." An analysis of Oregon crashes (*30*) found a counterintuitive effect between shoulder width and crashes – crashes increased with increased shoulder widths on freeways and decreased on non-freeway segments. In the literature review, the authors noted that three previous studies had found no relationship between shoulder width and crash frequency and one found crash frequencies lower on road segments with narrow shoulders. The authors noted that the risk homeostasis theory (behavior adapts to changes in perceived hazards) may explain that drivers are changing their behavior due to perceptions of reduced risks.

Trucks

A 2003 TxDOT project (*7*) reviewed design criteria with respect to trucks to determine if changes in the TxDOT *Roadway Design Manual* are recommended. Their final recommendations for shoulder widths were:

- increasing the outside shoulder width to 12 ft [3.6 m] along truck roadways and mixed flow roadways predicted to reach an annual average daily truck traffic (AADTT) of at least 5000 trucks per day during the design period;
- increasing the offset between the outer edge of the usable shoulder and vertical elements such as barriers by a minimum of 2 ft [0.6 m]; and
- paving all shoulders on high-volume truck routes, desirably to the same depth and composition as the mainlanes.

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

Table 6-2 lists suggested shoulder width values for high design speeds for two types of facilities. If a single set of values is preferred, then the values listed under "Truck Facility or High Truck Volumes" are suggested.

Table 6-2. Potential Shoulder Width Values for High Design Speed.

Shoulder	Passenger Car Facility		Truck Facility or High Truck Volumes [a]	
	4-lane divided	6-lane or more divided	4-lane divided	6-lane or more divided
Inside	10 ft [3.1 m]	10 ft [3.1 m]	12 ft [3.6 m]	12 ft [3.6 m]
Outside	10 ft [3.1 m]	10 ft [3.1 m]	12 ft [3.6 m]	12 ft [3.6 m]

[a] High truck volumes occur when the directional design hourly volume for truck traffic exceeds 250 veh/h.
Shaded areas reflect high-design-speed potential values.

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- Is there a relationship between shoulder width and speed for rural freeways? Do wider shoulders encourage higher speeds?
- Identify the relationship between shoulder width and safety for rural freeways. While research has shown that wider shoulders are associated with lower crash rates on rural two-lane highways, are wider freeway shoulders also associated with lower crash rates? How does the relationship change between the outside shoulder and the inside (median) shoulder?
- Are rumble strips recommended on high-speed roadways? If so, where should the rumble strips be placed (on the edgeline)? If not on the edgeline, how far from the edgeline?
- Previous research has stated that a wider outside shoulder is to be used when the average daily truck traffic is at least 5000 trucks per day during the design period. The *Green Book* indicates to use a wider shoulder when the directional design hourly volume for truck traffic exceeds 250 veh/h. Which of these values is the appropriate volume for a

high-design-speed roadway? Should the decision be made with the consideration of additional variables similar to the process developed and presented in the *Truck Facility Guidebook* (34)?

- Should the design criteria include discussion on enforcement areas? Should shoulders be wider in selected segments to assist with enforcement activities?

CHAPTER 7

PAVEMENT CROSS SLOPE

Pavement cross slope is essential for proper roadway drainage during precipitation events. The principal limit on the amount of cross slope used on pavement is driver comfort. The selection of the pavement cross slope also affects the "reverse crown" superelevation and minimum curve radius without using superelevation (see Chapter 8).

This chapter first discusses current United States guidance about pavement cross slope, both from drainage and drivability standpoints. Where appropriate, international guidance and information from other sources are included to enhance the discussion.

CURRENT GUIDANCE

This section outlines the current guidance used in the United States for selecting the pavement cross slope for roadways. In general terms, this guidance is universal, and is contained in a variety of sources, including the *Green Book* (*2*), AASHTO's interstate design standards (*9*), the TxDOT *Roadway Design Manual* (*1*), and the Texas *Hydraulic Design Manual* (*35*). However, there are some slight variations between the sources.

Pavement cross slope is required for adequate pavement surface drainage. If the cross slope is not steep enough, the water depth on the roadway may be deep enough to cover the top of the pavement surface texture, which in turn may cause hydroplaning (*36*). However, a steeply sloped pavement may be uncomfortable to drive on, and may cause vehicles to drift toward the edge of the pavement. Therefore, operational requirements and driver comfort limit the amount of cross slope that can be used.

In terms of cross slope selection, the *Green Book* states that:

> "cross slopes up to and including 2 percent are barely perceptible in terms of vehicle steering. However, cross slopes greater than 2 percent are noticeable and require a conscious effort in steering" (*2*).

As a result of this statement, the *Green Book* recommends that "high-type pavements" (i.e., Portland cement concrete and asphaltic concrete) should have a cross slope of 1.5 to 2 percent. The 1.5 percent minimum has been found to greatly reduce ponding on the roadway (*30*). In areas with "high precipitation intensities," the cross slope could be increased to 2.5 percent, but the use of a 2.5 percent cross slope should be limited to these areas only. The statements in the *Green Book* are not referenced, so it is unclear whether the finding that a 2 percent cross slope was "barely perceptible" came from research or is simply based on long-term experience. AASHTO's interstate design standards echo the cross slope values without the additional discussion (*9*).

The TRDM states that the "usual" cross slope value is 2 percent (*1*). For areas with high rainfall rates, steeper slopes may be used, referring the designer to the *Green Book*. The TRDM gives some additional guidance for situations where three or more lanes are being drained in the same direction. In this case, the outermost (i.e., right-most) lane(s) may have a slope 0.5 percent greater than the inside two lanes, and the inside two lanes may have a cross slope less than the normal 2 percent, typically 1.5 percent but not less than 1.0 percent.

The TRDM also discusses where the slope crown should be. Desirably, each paved section of roadway should have its crown in the middle. However, this creates a break point of 4 percent in the middle of the roadway, which could cause some vehicle control issues *(2)*. **At high speeds, the effect of this break point is not known**. For driver comfort, each direction of a divided highway may be sloped in the same direction, so the crown is at the inside edge of the pavement and would not be crossed by drivers whenever they change lanes.

Texas has a second source of information about roadway cross slope and drainage: the Texas *Hydraulic Design Manual* *(35)*. The *Hydraulic Design Manual* discusses roadway surface drainage and includes a specific section on hydroplaning. The *Hydraulic Design Manual* also presents specific equations for calculating the speed and water depth at which hydroplaning occurs. These equations were developed by Gallaway et al. *(36)*. Specific design values for roadway cross slope are not provided except for the "recommended" 2 percent cross slope on tangent sections. However, specific suggestions for reducing the likelihood of hydroplaning are provided, such as increasing the cross slope, using a permeable surface course (which allows the water to go through it, rather than collect on its surface), avoiding wheel path depressions greater than 0.2 in [5 mm], and use of transverse grooving in particularly troublesome areas. However, the *Hydraulic Design Manual* also includes the following cautionary statement:

> "Rainfall intensities can be so high in Texas that the designer cannot eliminate the potential for hydroplaning. Because rainfall intensities and vehicle speed are primary factors in hydroplaning, it is incumbent on the driver must [sic] be aware of the dangers of hydroplaning" *(35)*.

In light of this statement, the potential effects of hydroplaning for high-speed roadways deserve further investigation, especially with current tire designs and vehicle types. For example, pickup trucks may be more vulnerable to loss-of-control situations due to their rear-wheel drive configuration and relatively light rear weight when empty.

OTHER GUIDANCE

International guidance on cross slope generally centers on two values, 2 percent and 2.5 percent, without much variation. Germany uses a standard cross slope of 2.5 percent on their roadways, apparently without variation *(37)*. No other cross slope values are mentioned in the design guidance. In addition, a single cross slope on each direction of pavement is used on divided highways, also apparently without variation. No other cross slope configuration is illustrated for divided highways. France *(38)*, Sweden *(39)*, and the United Kingdom *(40)* also use a 2.5 percent cross slope as their standards.

Spain's cross slope design is generally very similar to Germany's *(41)*. The only major difference is the use of a 2 percent standard cross slope, like U.S. standards, instead of 2.5 percent.

Canada's design standards state different cross slope minimums for different pavement types: 2 percent for asphalt concrete and 1.5 percent for Portland cement concrete *(42)*. The maximum cross slope is not directly stated. However, the guidelines for resurfacing indicate that the "acceptable tolerance" ranges up to 2.5 percent for a design speed of 75 mph [120 km/h], so 2.5 percent would probably be the maximum cross slope for higher design speeds.

Minimum design criteria and guidelines were developed for TxDOT for use on the TTC-35 high-priority corridor *(11)*. Their recommendations for the cross slope across the entire roadway is 2 percent minimum.

DISCUSSION

Hydroplaning

Useful literature on acceptable cross slopes is relatively sparse, especially after 1980. Gallaway et al. performed several studies in the mid to late 1970s investigating the effects of roadway cross slope on hydroplaning *(35)*. Hydroplaning is dependent on several factors: speed, tire pressure, roadway surface texture, water depth (which in turn is a function of rainfall intensity), tire contact area, and tread depth. They developed several equations for predicting the speed and water depth when hydroplaning would occur. These equations are also presented in the Texas *Hydraulic Design Manual* (*35*). For brevity's sake they are not repeated here. The *Hydraulic Design Manual* includes a caveat that the Gallaway et al. equations are only useful for normal highway speeds below 55 mph [88 km/h]. Gallaway et al. indicate no such restriction in their report and apparently tested speeds up to 65 mph [105 km/h]. However, the National Maximum Speed Limit of 55 mph [88 km/h] was in force at the time of the research, and this factor may be the source of the caveat.

Gallaway et al. offered several solutions for hydroplaning, and these are somewhat different from the ones presented in the *Hydraulic Design Manual*. A coarser surface texture prevents water from covering the tops of the aggregate, which preserves traction. A high traction overlay is used in Texas when wet weather traction is an issue. However, having large aggregate at the surface of a mix can cause pop-outs, and flying rock may be even more an issue at 85 mph [137 km/h] than it is at 70 mph [112 km/h]. Also, the *Hydraulic Design Manual* points out that a coarser surface texture impedes pavement drainage *(35)*. The coarser surface also causes more tire wear, and may wear out quickly itself. A second option is increasing the cross slope. A third is relocating the crown to reduce the length of the drainage path.

Huebner et al. *(43)* also investigated hydroplaning. They concluded that Gallaway et al.'s equations were adequate for water depths of 0.095 in [2.4 mm] or greater, but that hydroplaning speeds are better described by another equation at smaller depths. The equation Huebner et al. developed for water depths below 0.095 in [2.4 mm] is:

$$V_h = 26.04 D_w^{-0.259} \quad \textbf{(7-1)}$$

Where:

V_h = hydroplaning speed, mph; and
D_w = water depth, inches.

Visibility

Visibility during precipitation events is another related concern, because the combination of increased vehicle speed, precipitation intensity, wind speed and direction, and spray from other vehicles impairs driver visibility at the same time the wet pavement may increase stopping distances. Ivey et al. investigated the relationship between visibility and rainfall intensity (*44*). Figure 7-1 shows the relationship between speed, stopping sight distance, and available sight distance for rainfall intensities of 1 inch/h [25.3 mm/h], 2 inch/h [50 mm/h], and 3 inch/h [75 mm/h]. Figure 7-2 illustrates the relationship of rainfall intensity, rainfall duration, and return interval for Houston, Texas. Notice that a 2 year return interval (i.e., 50 percentile frequency), 30 minute duration storm has an intensity of 3 inch/h [75 mm/h], and that rarer storms may have brief intensities as high as 8 inch/h [203 mm/h]. Houston receives more rainfall than most parts of Texas. However, even drier portions of the state such as El Paso, San Antonio, Midland, and Lubbock may experience equally high rainfall rates for brief periods in the cores of thunderstorms.

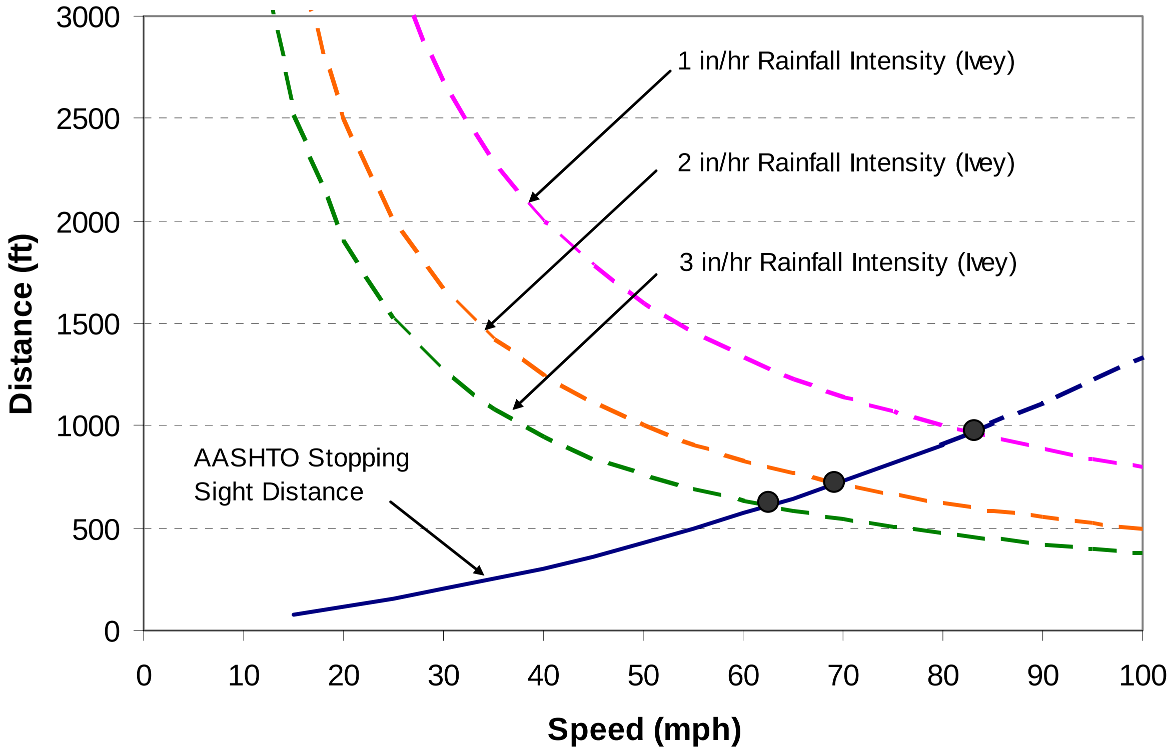
As Figure 7-1 shows, the available sight distance during a 1 inch/hr [25.3 mm/h] rain event drops below the required stopping sight distance at 83 mph [134 km/h], at about 70 mph [112 km/h] for a 2 inch/hr [50 mm/h] event, and at about 62 mph [100 km/h] for a 3 inch/hr [75 mm/h] event. Drivers behaving in a reasonable and prudent manner are assumed to slow during these events. However, a 1 inch/hr [25.3 mm/h] event may not be considered to be severe enough to cause most drivers to slow down.

Vehicle Spray

The visibility values shown in Figure 7-1 do not include the effects of spray from other vehicles. Spray is another source of reduced visibility, especially in the vicinity of heavy vehicles. Roadway spray can be reduced to some extent by providing an adequate drainage or a porous surface course, thereby removing water from the roadway. However, some elements of surface design intended to improve traction may retain more water on the roadway surface rather than less (e.g., coarser surface textures).

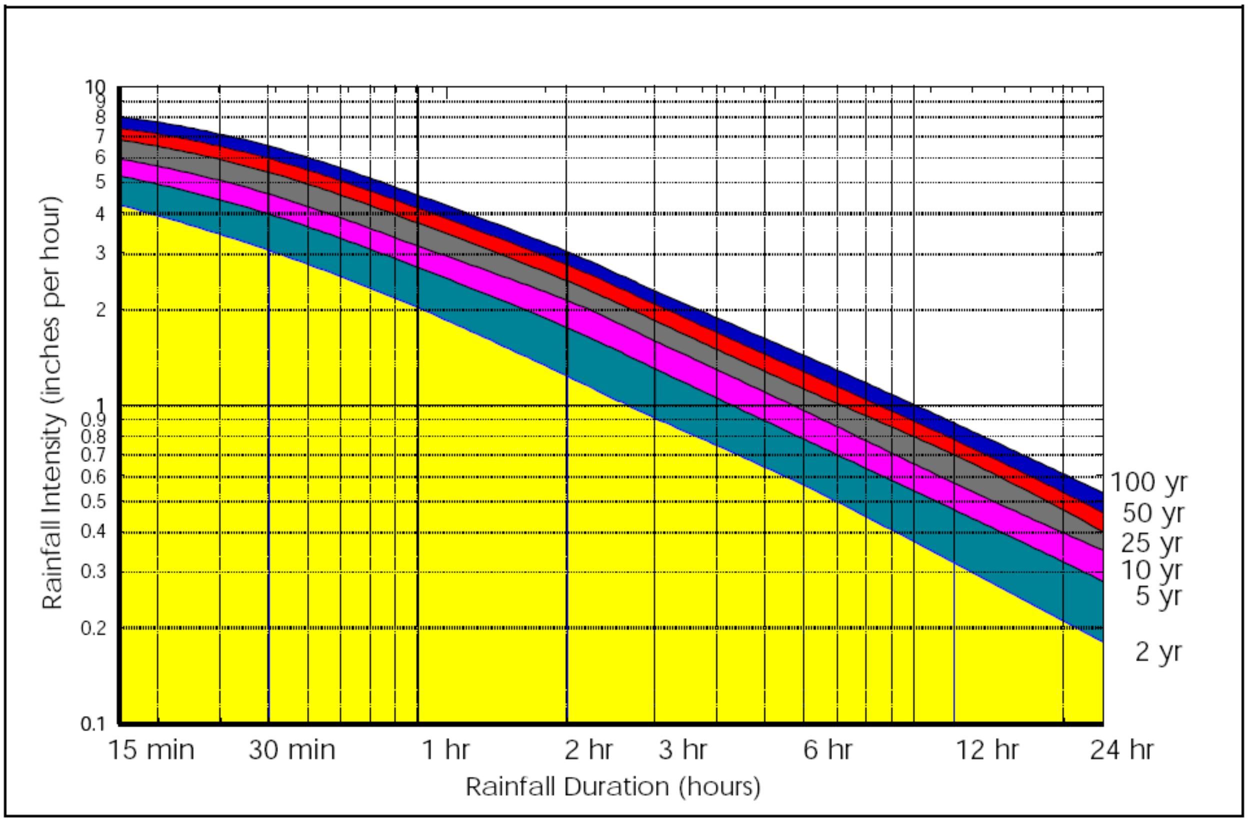
Crashes

Dunlap et al. (*45*) found that crashes on high-speed roadways depended on both degree of curvature and grade. However, the conditions of curvature and grade that caused problems varied from location to location depending on weather conditions and tire conditions. Also, both analytical and computer simulation indicated that grade was not a major contributor to crashes in and of itself. Instead, surface drainage was an important consideration on curves. Specifically, a surface drainage problem on a section of the Ohio Turnpike, combined with worn tires on some vehicles, created an abnormal crash pattern. This potential problem may be amplified with increased speeds, increasing the need for new research on vehicle control issues at high speeds.



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 7-1. Relationship between Speed and Visibility Distance with Increasing Rainfall Intensity (Data for Figure from Ivey et al. (*44*)).



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 7-2. Rainfall Intensity-Duration-Frequency Curve for Houston, Texas (Data for Figure from Ivey et al. (*44*)).

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

Based on the guidance provided by the *Green Book*, TRDM, *Hydraulic Design Manual*, and other sources, the current guidance contained in the TRDM for roadway cross slope appears to be adequate. Some issues may arise with cross slopes of 3 percent or higher, and these should be avoided. However, based on drainage requirements, a cross slope of up to 2.5 percent may be used for the right-most lanes of a three or more lane cross section. Cross slopes flatter than 1.5 percent should be avoided to reduce ponding.

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- The potential effect of hydroplaning on the safety of a high-speed roadway deserves further investigation, especially with more current tire designs and vehicle types. Common configurations of vehicles should be tested, such as empty pickup trucks, sport utility vehicles, front-wheel drive cars, and lightweight vehicles.

- The potential effects of cross slope on driver reactions (e.g., steering) should be investigated. What is the effect of a steeper cross slope on driver behavior, especially at high speeds? These effects will need to be weighed against any drainage improvements that may result from the increased cross slope.
- The effects of traversing breakpoints of 4 percent or more at high speeds, which would be required if a roadway crown is placed in the middle of a travel direction, is not known. Additionally, the maximum slope break allowed between the travel lanes and the shoulder is 7 percent, and the effect of a break this size on driver reactions at high speeds is also not known.

CHAPTER 8

HORIZONTAL ALIGNMENT AND SUPERELEVATION

Horizontal alignment and superelevation are related so closely that the topics were combined to provide a more meaningful and logical discussion. The normal pavement cross slope (Chapter 7) also affects both the transition to superelevation and the minimum curve radius without using additional superelevation.

CURRENT GUIDANCE

This section outlines the current guidance used in the United States for superelevation for horizontal curvature, as well as the guidance for superelevation transition from the *Green Book* (*2*) and the TxDOT *Roadway Design Manual* (*1*).

It is important to note that at the time of this writing, the TRDM uses superelevation information from a prior edition of the *Green Book*. The TRDM is currently being modified to agree with the guidance in the current edition of the *Green Book*. Until this process is complete, the TRDM and *Green Book* will differ in some aspects. The 2004 *Green Book* has revised procedures for horizontal curve design compared to earlier editions. However, these revisions were made for low-speed (i.e., less than 50 mph [80 km/h]) facilities, so curve and superelevation design for high-speed roadways were not affected by the revisions.

Determination of Curve Radius

For any given design speed, the radius of a curve can be found using the following equation:

$$R = \frac{V^2}{C(e+f)} \tag{8-1}$$

Where:

> R = curve radius (ft or m),
> V = design speed (mph or km/h),
> C = conversion constant (15 for US customary units, 127 for metric),
> e = curve superelevation, and
> f = available side friction factor.

The curve radius is a function of the sum of the superelevation and the available side friction, which are the outward component of the forces acting on the vehicle on a curve. For any curve, it is theoretically possible for a vehicle's path to be determined by only side friction, or by only superelevation, or by some combination of the two.

Superelevation

Superelevation is determined based on the "comfortable" lateral acceleration imparted on a vehicle traversing a horizontal curve. "Comfortable," in this case, is driver and occupant comfort. A cornering vehicle undergoes acceleration inward toward the center of the curve, which is felt by the vehicle's occupants as an "outward force." To maintain a constant speed on a curve, a combination of vehicle side friction and superelevation are required. If a curve is superelevated so that the required side friction is zero, then theoretically no steering inputs would be required for a vehicle traveling a certain speed, and the vehicle would stay on the roadway regardless of the curvature. A vehicle traveling faster or slower than this speed would require both steering inputs and tire friction to prevent it from leaving its path.

Because the maximum side friction is a critical assumption in roadway design, it is discussed as a separate topic.

Side Friction

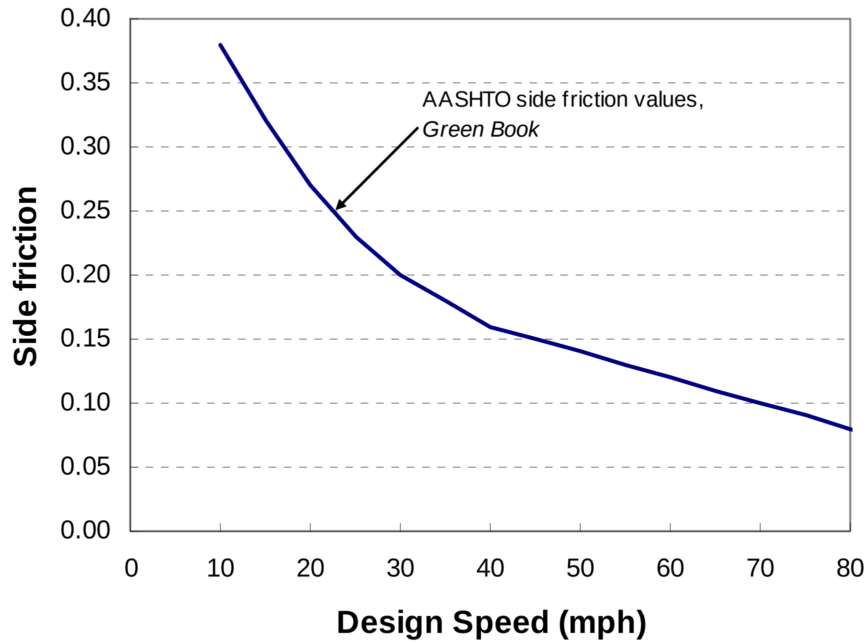
The side friction is the amount of tire-to-pavement friction that is available for use by a turning vehicle. The amount of friction available for turning has been found to vary with speed. However, the use of all available friction in turning maneuvers can result in swerving, vehicle drift, and increased steering effort. In those situations, a driver's level of concentration increases, and his or her cone of vision narrows.

Because vehicle occupants rarely will accept the use of all available friction in normal operation, a separate, lower value of maximum side friction is used that corresponds with occupant comfort levels. Various research efforts have found that the available side friction varies with vehicle speed from approximately 0.14 at 50 mph [80 km/h] to no more than 0.10 at 70 mph [112 km/h] (*2*). The *Green Book* uses a linear relationship for maximum side friction at speeds above 50 mph [80 km/h]. This relationship reduces the side friction factor by 0.01 for each 5 mph [8 km/h] of speed over 50 mph [80 km/h], and is shown graphically in Figure 8-1. This linear relationship is assumed to continue to at least 80 mph [128 km/h]. The side friction distribution method was changed in the 2004 *Green Book*, but the changes were for speeds below 50 mph [80 km/h] and have no effect on typical rural highway design speeds.

Superelevation Distribution

Distributing the amount of side friction and superelevation used on a horizontal curve is very important in the design of curves with radii greater than the minimum radius. The *Green Book* provides five different methods for using a combination of side friction and superelevation for each curve design (*2*), as illustrated in Figure 8-2:

- Method 1 assumes that superelevation and side friction are directly proportional to the inverse of the radius, which produces the straight line shown in Figure 8-2.
- Method 2 assumes that all of the available side friction is used first, and then superelevation is used to achieve higher speeds (up to the maximum superelevation).



Note: 1 mph = 1.61 km/h

Figure 8-1. AASHTO Side Friction Values for Horizontal Curve Design (*2*).

- Method 3 is the opposite of Method 2: superelevation is used first unless the maximum allowable superelevation is reached, and then side friction alone will produce higher speeds on curves up to the maximum side friction demand.
- Method 4 is essentially the same as Method 3, except it uses average running speed instead of design speed.
- Method 5 is a parabolic distribution that uses more superelevation than side friction at large radii and the opposite as the curve radii approaches the minimum. Conceptually, Method 5 is similar to the layout of a vertical curve that has a different grade on each side of the curve.

Methods 2 and 3 form obvious boundary conditions for superelevation distribution.

For high-speed roadways, the *Green Book* uses Method 5. While the use of Method 5 is near-universal in the United States, the calculations to apply it are complicated. The algorithm for Method 5 is listed in Figure 8-3 and illustrated in Figure 8-4. Conceptually, the algorithm in Figure 8-3 is similar to the calculations used to design a sag vertical curve.

Method 5 requires a curve running speed as well as a side friction factor. The running speed is believed to have been based on an assumption that drivers will slow down during inclement weather. The values of running speed compared to design speed are shown in Figure 8-5. Notice that the running speed's rate of increase changes at 60 mph [96 km/h], and that the amount of speed difference is assumed to increase as the design speed increases.

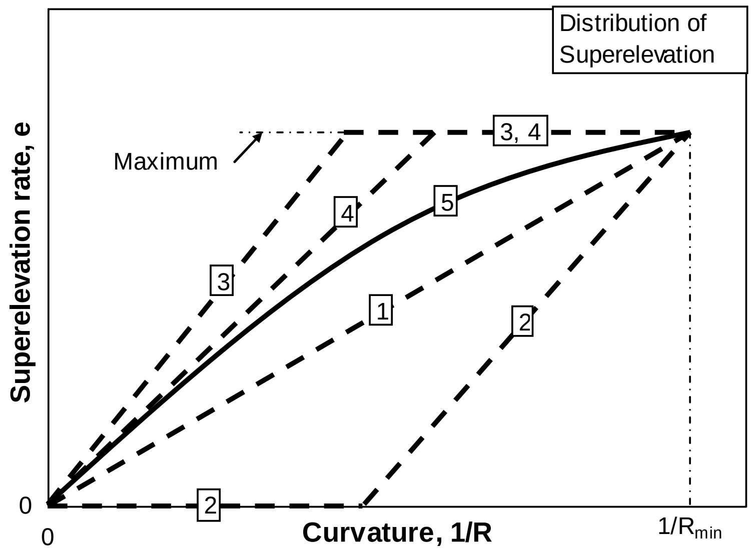


Figure 8-2. Illustration of Superelevation Distribution Methods (*2*).

(Metric)

$$0.01e + f = \frac{V_D^{\ 2}}{127R}$$

Where:
- V_D = design speed, km/h;
- e = superelevation, percent;
- e_{max} = maximum superelevation, percent;
- f = side friction factor;
- f_{max} = maximum allowable side friction factor;
- R = curve radius, m;
- R_{min} = minimum curve radius, m;
- R_{PI} = curve radius at the point of intersection (PI) of legs 1 and 2 of the f distribution curve; and
- V_R = running speed, km/h.

$$R_{\min} = \frac{V_D^2}{127\left(0.01e_{\max} + f_{\max}\right)}$$

and

$$R_{PI} = \frac{V_R^2}{127\left(0.01e_{\max}\right)}$$

$\left(0.01e + f\right)_D - \left(0.01e + f\right)_R = h$, so at R_{PI}:

$$h_{PI} = \left(\frac{\left(0.01e_{\max}\right)V_D^2}{V_R^2}\right) - 0.01e_{\max}$$

(US Customary)

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

Where:
- V_D = design speed, mph;
- e = superelevation, percent;
- e_{max} = maximum superelevation, percent;
- f = side friction factor;
- f_{max} = maximum allowable side friction factor;
- R = curve radius, ft;
- R_{min} = minimum curve radius, ft;
- R_{PI} = curve radius at the point of intersection (PI) of legs 1 and 2 of the f distribution curve; and
- V_R = running speed, mph.

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

and

$$R_{PI} = \frac{V_R^2}{15\left(0.01e_{\max}\right)}$$

$\left(0.01e + f\right)_D - \left(0.01e + f\right)_R = h$, so at R_{PI}:

$$h_{PI} = \left(\frac{\left(0.01e_{\max}\right)V_D^2}{V_R^2}\right) - 0.01e_{\max}$$

Figure 8-3. Method 5 for Superelevation Distribution (*2*).

(Metric)	(US Customary)
Where h_{PI} = PI offset from the $1/R$ axis (see Figure 8-4).	Where h_{PI} = PI offset from the $1/R$ axis (see Figure 8-4).
Also, $S_1 = h_{PI}(R_{PI})$	Also, $S_1 = h_{PI}(R_{PI}) / 5729.58$
Where S_1 = slope of leg 1, and $S_2 = \dfrac{f_{max} - h_{PI}}{\dfrac{1}{R_{min}} - \dfrac{1}{R_{PI}}}$	Where S_1 = slope of leg 1, and $S_2 = \dfrac{f_{max} - h_{PI}}{5729.58\left(\dfrac{1}{R_{min}} - \dfrac{1}{R_{PI}}\right)}$
Where S_2 = slope of leg 2.	Where S_2 = slope of leg 2.
The middle ordinate MO of the f distribution curve is: $MO = \dfrac{L_1 L_2 (S_2 - S_1)}{2(L_1 + L_2)}$	The middle ordinate MO of the f distribution curve is: $MO = \dfrac{L_1 L_2 (S_2 - S_1)}{2(L_1 + L_2)}$
Where: $L_1 = 1/R_{PI}$ and $L_2 = 1/R_{min} - 1/R_{PI}$.	Where: $L_1 = 5729.58/R_{PI}$ and $L_2 = 5729.58(1/R_{min} - 1/R_{PI})$.
Substituting L_1 and L_2 into the MO equation gives: $MO = \dfrac{1}{R_{PI}}\left(\dfrac{1}{R_{min}} - \dfrac{1}{R_{PI}}\right)\left(\dfrac{S_2 - S_1}{2}\right)R_{min}$	Substituting L_1 and L_2 into the MO equation gives: $MO = \dfrac{5729.58}{R_{PI}}\left(\dfrac{1}{R_{min}} - \dfrac{1}{R_{PI}}\right)\left(\dfrac{S_2 - S_1}{2}\right)R_{min}$
For any curve radius R: $(0.01e + f)_D = \dfrac{(0.01e_{max} + f_{max})R_{min}}{R}$	For any curve radius R: $(0.01e + f)_D = \dfrac{(0.01e_{max} + f_{max})R_{min}}{R}$
For $1/R \le 1/R_{PI}$, the f distribution between zero and $1/R_{PI}$, f_1, is: $f_1 = MO\left(\dfrac{R_{PI}}{R}\right)^2 + \dfrac{S_1}{R}$	For $1/R \le 1/R_{PI}$, the f distribution between zero and $1/R_{PI}$, f_1, is: $f_1 = MO\left(\dfrac{R_{PI}}{R}\right)^2 + \dfrac{5729.58S_1}{R}$
And the corresponding superelevation, e_1, is: $0.01e_1 = (0.01e + f)_D - f_1$	And the corresponding superelevation, e_1, is: $0.01e_1 = (0.01e + f)_D - f_1$
For $1/R > 1/R_{PI}$, the f distribution between $1/R_{PI}$ and $1/R_{min}$, f_2, is: $f_2 = MO\left(\dfrac{\dfrac{1}{R_{min}} - \dfrac{1}{R}}{\dfrac{1}{R_{min}} - \dfrac{1}{R_{PI}}}\right) + h_{PI} + S_2\left(\dfrac{1}{R} - \dfrac{1}{R_{PI}}\right)$	For $1/R > 1/R_{PI}$, the f distribution between $1/R_{PI}$ and $1/R_{min}$, f_2, is: $f_2 = MO\left(\dfrac{\dfrac{1}{R_{min}} - \dfrac{1}{R}}{\dfrac{1}{R_{min}} - \dfrac{1}{R_{PI}}}\right) + h_{PI} + 5729.58S_2\left(\dfrac{1}{R} - \dfrac{1}{R_{PI}}\right)$
And the corresponding superelevation, e_2, is: $0.01e_2 = (0.01e + f)_D - f_2$	And the corresponding superelevation, e_2, is: $0.01e_2 = (0.01e + f)_D - f_2$

Figure 8-3 (continued). Method 5 for Superelevation Distribution (2).

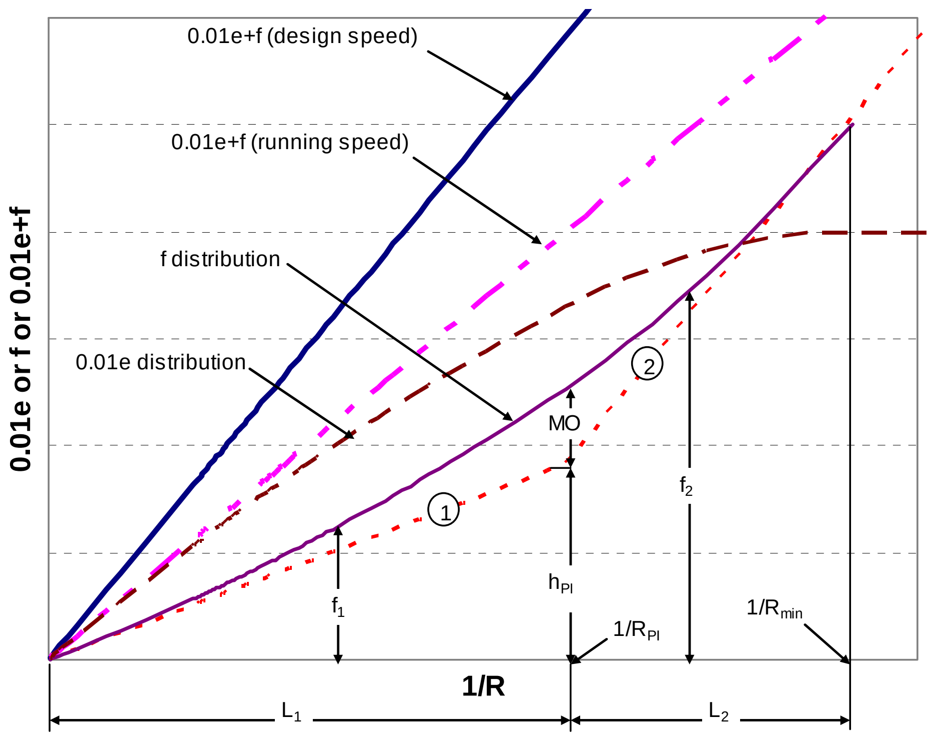
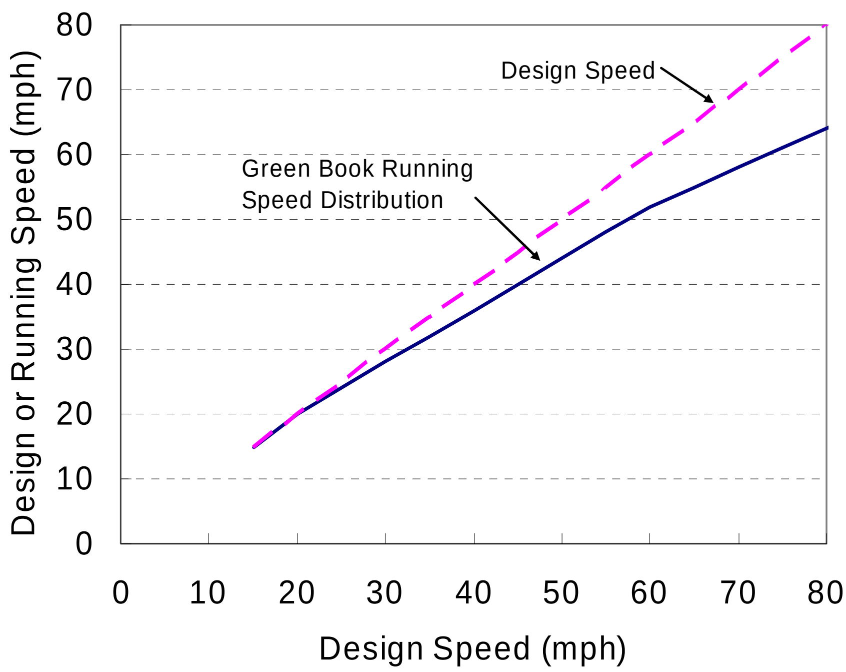


Figure 8-4. Illustration of Method 5 Components, Reproduced from *Green Book* Exhibit 3-18 (2).



Note: 1 mph = 1.61 km/h

Figure 8-5. *Green Book* Design Speed versus Running Speed (2).

Maximum Superelevation Rate

TRDM (*1*) recommends maximum superelevation rates of 6 to 8 percent statewide, depending on the prevalence of icing conditions. This guidance is consistent with guidance in the *Green Book* (*2*).

Superelevation Transition

The superelevation transition methods used by TRDM and the *Green Book* are similar. However, there are important differences between the two that deserve attention.

The *Green Book*'s superelevation transition method is shown in Figure 8-6. There are essentially four regions in the figure: normal crown, tangent runout, runoff, and full superelevation. Tangent runout is the distance required to rotate the outside lane(s) of the cross section to horizontal (i.e., from A to B in Figure 8-6). Runoff is the distance required to rotate the cross section from the end of the runout to full superelevation. For divided highways with a constant cross slope, there is no runout. The runoff rate in the *Green Book* is constant, although the sharp, angular breakpoints should be "appropriately rounded off" (*2*).

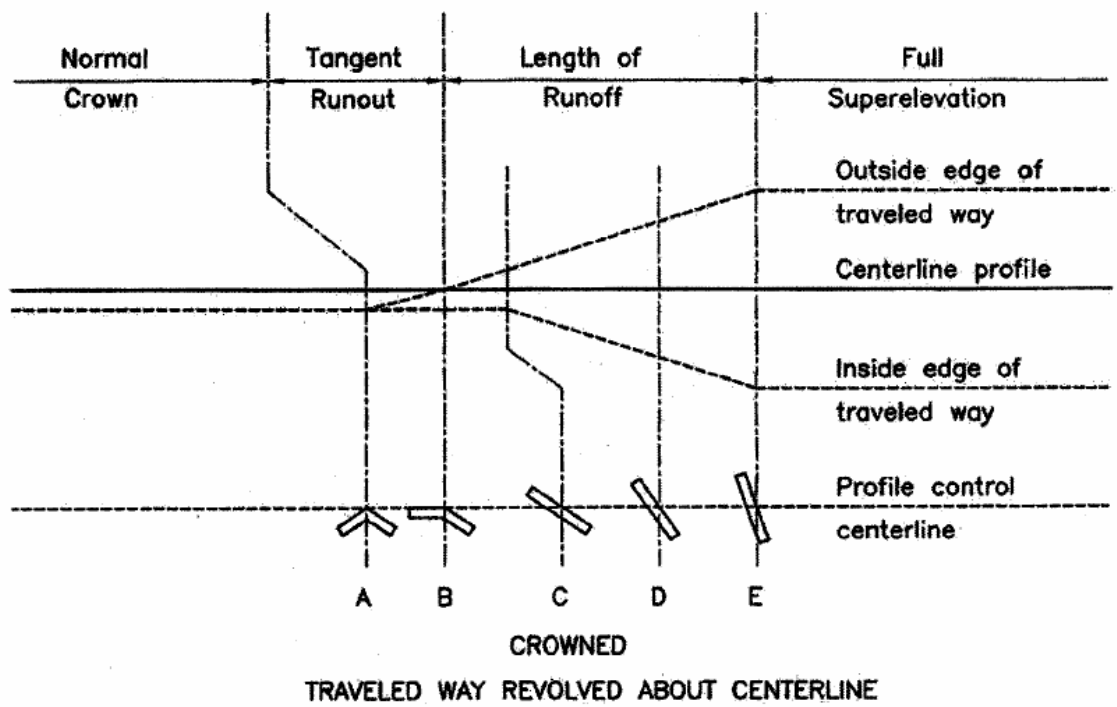


Figure 8-6. *Green Book* Method of Superelevation Transition (*2*).

To find the length of runoff (i.e., from B to E in Figure 8-6), the *Green Book* uses the following equation:

$$L_r = \frac{(wn_1)e_d}{\Delta}(b_w) \qquad (8\text{-}2)$$

Where:

L_r = minimum length of superelevation runoff,
Δ = maximum relative gradient,
n_1 = number of lanes rotated,
b_w = adjustment factor for number of lanes rotated,
w = width of one traffic lane, and
e_d = design superelevation rate (i.e., for that curve).

The adjustment factor b_w is 1.0 when one lane is rotated, and increases by 0.5 for each lane rotated (i.e., for two rotated lanes, b_w = 1.5, for three rotated lanes, b_w = 2.0, etc.).

TRDM Figure 2-3 shows the superelevation transition design method for Texas (*1*). Generally, the figure looks similar to Figure 8-6; however, there are four important differences. First, the rate of rotation changes along the length of the runoff, which is indicated by the reverse curve instead of a straight line. The second and more important difference involves tangent runout and runoff. The TRDM figure shows no tangent runout. Instead, the entire length is simply a "superelevation transition," without differentiating between runout and runoff. In other words, the TRDM method proceeds directly from A to E in Figure 8-6 without distinguishing between "runout" and "runoff."

Additionally, TRDM calculates the length of superelevation transition using the following equation (*1*):

$$L_{CT} = \frac{(CS)(W)}{G} \qquad (8\text{-}3)$$

Where:

L_{CT} = calculated length of superelevation transition,
CS = percent change in cross slope of superelevated pavement,
W = width of pavement sections to be rotated, and
G = maximum relative gradient.

Superficially, the TRDM equation is much different (and simpler) than that from the *Green Book*. The obvious difference is that there is no adjustment factor for the number of lanes rotated in the TRDM expression. For a single rotated lane, the two methods are identical. If more than one lane is being rotated, the TRDM method provides for a shorter superelevation transition area than the *Green Book*'s runoff. Moreover, the *Green Book* also includes tangent runout, which TRDM does not. It is not clear whether the differences will be important at design speeds above 80 mph [129 km/h]. Further research is necessary.

A third difference exists between the superelevation transition methods of the *Green Book* and TRDM. TRDM has a minimum transition length requirement of 2 seconds of travel time, depending on the design speed, for appearance purposes. The *Green Book* has dropped the

2 second length rule for transition appearance, stating that it was not necessary for operational reasons.

Finally, the *Green Book* and TRDM differ on the amount of superelevation transition that occurs prior to the start of a horizontal curve. The *Green Book*'s amount of superelevation runout on the tangent prior to the curve varies by the design speed of the curve and the number of lanes being superelevated. The proportion of superelevation runoff on the tangent prior to the curve (for design speed higher than 50 mph [80 km/h]) varies from 0.7 for one lane to 0.85 for three lanes. TRDM simply states that two-thirds (0.67) of the transition is on the tangent and the remainder on the curve.

Currently, vehicle and driver performance data are lacking on superelevation transition sections for design speeds above 80 mph [128 km/h]. It is possible that vehicles require more transition time to prevent undesirable lateral accelerations in the transition area. Trucks and other vehicles with high centers of gravity may also need more transition distance to prevent load shifts and rollovers.

Maximum Relative Gradient

Both the *Green Book* and TRDM use the same values for maximum relative gradient for superelevation transition, although the *Green Book*'s value is for the gradient of the edge of the traveled way and TRDM's value is only for calculating the transition length. These values are shown in Table 8-1. Above 50 mph [80 km/h], the rate of reduction of the maximum relative gradient is 0.025 percent for each 5 mph [8 km/h] of increasing design speed.

Table 8-1. Maximum Relative Gradients for Superelevation Transition (*1*,*2*).

(US Customary)			(Metric)		
Design Speed (mph)	**Maximum Relative Gradient (%)[a]**	**Equivalent Maximum Relative Slope**	**Design Speed (km/h)**	**Maximum Relative Gradient (%)[a]**	**Equivalent Maximum Relative Slope**
15	0.78	1:128	20	0.80	1:125
20	0.74	1:135	30	0.75	1:133
25	0.70	1:143	40	0.70	1:143
30	0.66	1:152	50	0.65	1:150
35	0.62	1:161	60	0.60	1:167
40	0.58	1:172	70	0.55	1:182
45	0.54	1:185	80	0.50	1:200
50	0.50	1:200	90	0.47	1:213
55	0.47	1:213	100	0.44	1:227
60	0.45	1:222	110	0.41	1:244
65	0.43	1:233	120	0.38	1:263
70	0.40	1:250	130	0.35	1:286
75	0.38	1:263			
80	0.35	1:286			

[a] Maximum relative gradient for profile between edge of traveled way and axis of rotation.

Horizontal Sight Distance

The horizontal sight distance on curves is very important for safe operation. Both the *Green Book* and TRDM present the same method for determining horizontal sight distance. In the case of high-speed roadways, the same procedure can be used. It is likely that the only obstructions to horizontal sight distance for these roadways will be safety appurtenances (e.g., median barrier). The sight distance impact of these objects can be minimized through the appropriate design and placement of the devices.

Drainage

Superelevation transition inevitably produces points where the pavement cross slope is zero. These points can cause water to pond during rain, which in turn can lead to hydroplaning and loss of control. Because hydroplaning is a function of speed, the problem will be magnified at design speeds above 80 mph [128 km/h]. Therefore, drainage becomes very important in the design, and ponding must be avoided if at all possible. The *Green Book* suggests locating these zero points on a grade of 0.5 percent or greater to allow the water to move laterally along the roadway to a point with greater cross slope (*2*). Additionally, pavement treatments such as porous surface courses may be used to improve drainage and prevent ponding.

OTHER GUIDANCE

International guidance about superelevation rates varies somewhat by country but is reasonably consistent within a broad range of values. For example, Germany specifies a maximum superelevation rate of 8 percent (*37*). This guidance is similar to guidance in the *Green Book*. Other countries typically range from 6 to 8 percent, depending on climate and other factors. (*10, 38, 39, 40, 41, 42, 46*)

A few countries have more unique superelevation limits. For example, Spain uses a maximum superelevation of 8 percent for design speeds up to 75 mph [120 km/h] (*41*). However, as the design speed increases above 75 mph [120 km/h], the maximum superelevation *decreases* to 4.3 percent at 95 mph [153 km/h]. The design guidelines do not provide a reason for this reduction, although the amount of side friction assumed to be available may explain this decrease.

As another example, Australia uses different maximum superelevation rates for different terrain types (*46*). For flat terrain, the maximum superelevation is 6 percent, although higher superelevation is allowed for "low [small] radius curves." In "general" terrain, the maximum superelevation is 10 percent, and in mountainous terrain, the maximum superelevation is 12 percent. In the latter two terrain types, there are maximum design speeds as well: "general" at 68 mph [110 km/h], and mountainous at 56 mph [90 km/h].

Side Friction

Table 8-2 shows the design side friction factors for roadways with a 75-mph [120 km/h] design speed. The values are fairly consistent for the various countries, ranging from a low of 0.06 (Japan) to 0.12 (Ireland), with most everyone else around 0.09 or 0.10. However, the assumptions for higher design speeds (if those speeds are used) were not provided. The general agreement on the available side friction for curvature appears to support the *Green Book*'s distribution, as shown in Figure 8-1.

Table 8-2. Side Friction Factors for Various Countries at 75 mph [120 km/h] (*10*, *37, 38, 39, 40*, *41, 42*).

Country	Maximum Side Friction at 75 mph [120 km/h]
Australia	0.11
Austria	0.10
Belgium	0.10
Canada	0.09
France	0.10
Germany	0.07
Ireland	0.12
Italy	0.10
Japan	0.06
Luxembourg	0.10
The Netherlands	0.08
Portugal	0.10
Spain	0.10
Switzerland	0.10
United Kingdom	0.09
United States	0.09

Superelevation Distribution

Bonneson *(47)* presented some international guidance on superelevation and side friction factor distribution as part of NCHRP Report 439. Many of the countries he surveyed used a linear superelevation distribution method (i.e., similar to Method 1 in Figure 8-2). The use of a linear relationship also effectively eliminates the need for large curve design tables such as *Green Book* Exhibits 3-25 through 3-29, and is considerably different from current U.S. practice. Because the United States is the only country to use a distribution method like Method 5, international practice for superelevation distribution does not directly relate to roadway design practices in Texas.

TTC-35

Minimum design criteria and guidelines were developed for TxDOT for use on the TTC-35 high-priority corridor *(11)*. Their recommendations for horizontal alignment (with an assumed 80 mph [128 km/h] design speed on mainlanes) are:

- Minimum radius – 4605 ft [1405 m], and
- Maximum superelevation – 0.06 ft/ft.

DISCUSSION

The minimum radius curve for a particular superelevation rate e_{max} and speed V can be found using the following equation:

$$R_{\min} = \frac{V^2}{C\left(e_{\max} + f_{\max}\right)} \tag{8-4}$$

Where:

R_{min} = minimum curve radius (ft or m),
V = design speed (mph or km/h),
C = conversion constant (15 for US customary units, 127 for metric),
e_{max} = maximum superelevation rate, and
f_{max} = maximum available side friction factor.

The *Green Book* applies this equation, along with the others shown in Figure 8-3 for Method 5, to create tables of horizontal curve radii and superelevation runoff values for each design speed and maximum superelevation *(2)*. Therefore, the same method will be applied here. Because Method 5 is used consistently in the United States, its use is justified at higher design speeds for the sake of consistency.

Bonneson *(47)* found that any superelevation distribution method that exists between the lines for Method 2 and Method 3 in Figure 8-2 would be satisfactory from an operational standpoint. He proposed his own superelevation distributions based on that finding. These distributions were not adopted by AASHTO.

Side Friction for Curves with High Design Speeds

Because the side friction values provided in the *Green Book* are for 80 mph [128 km/h] or less, side friction factors must be extrapolated for use at higher speed. The side friction relationship shown in Figure 8-1 has been in use for many years, although there are two main concerns regarding it. First, the relationship was created using a single data point at 70 mph [112 km/h] (referenced in the *Green Book* to the Stonex and Noble study on the Pennsylvania Turnpike in 1940). Data for higher speeds are absent, so the shape of the relationship may not necessarily remain linear beyond 70 mph [112 km/h]. Also, there is only a single point representing the current 70-mph [112 km/h] to 80-mph [128 km/h] design speed range.

The second concern relates to the dates the studies were performed. The Stonex and Noble study establishing the 0.10 side friction for 70 mph [112 km/h] was published in 1940, shortly after the Pennsylvania Turnpike (the first long-distance freeway in the United States) opened. The other studies cited in the *Green Book* took place between1936 and 1960. Vehicle design, pavement design, tire design, and driver tendencies have all changed considerably since these studies were performed. It is not clear whether the values shown in the *Green Book* are still appropriate. **Further research is needed to determine whether the assumed side friction distribution is correct for speeds above 80 mph [130 km/h].**

One of the most recent studies of superelevation rates was performed by Bonneson (*47*). He analyzed vehicle performance for a wide range of speeds and curves and determined the friction values based on 95th percentile operating speed and the design speed of the curve. The *Green Book* illustrates the findings from different side friction studies (see Figure 8-8) and states that the side friction values found by Bonneson are "similar" to the values previously in use (*2*). The values assumed for high-speed design in the *Green Book* are shown as a solid, bold line in Figure 8-7.

Recently, Tan tested the AASHTO side friction values on a test track at Penn State University (*48*). She asked vehicle passengers to rate whether they were comfortable or uncomfortable when driven around a test track at various speeds. The passengers reported their comfort observations during tests with the passengers blindfolded and unrestrained (i.e., no seat belts), not blindfolded and not restrained, and not blindfolded and restrained. Vehicle speeds used in the tests did not exceed 55 mph, so no information was available for high design speeds.

Overall, Tan observed that the AASHTO side friction values were reasonable for vehicle passenger comfort as long as the vehicle did not exceed the design speed of the curve and the curve's design speed was below 55 mph. However, Tan observed that these values are also conservative, and that passengers appeared to be willing to accept higher side lateral accelerations than were provided in the *Green Book*. Tan made some other observations that reduce the value of this finding. First, she observed that passenger comfort and driver comfort were not equivalent. The driver of the vehicle is in control of vehicle tracking and speed, and the driver may be more accepting of sharper curvature than a passenger who does not have a similar level of control. Second, Tan observed that the difference between the blindfolded and unblindfolded tests were too small to be practically significant. Moreover, because a driver may have a higher discomfort threshold and will not be blindfolded while driving, the results of tests on the comfort of blindfolded passengers are not necessarily closely related to real-world conditions. Finally, because of the driver's higher discomfort level compared to a passenger's, the side friction values may be very conservative for a driver. Tan stated that future observations about comfort on horizontal curvature should use vehicle drivers, not passengers (*48*).

Without information about the amount of side friction demand at speeds above 80 mph [128 km/h], the linear relationship for side friction was assumed to stay constant and was extrapolated to 100 mph [161 km/h] (see Figure 8-8). If the linear extrapolation shown in Figure 8-8 is extended to 120 mph [190 km/h], the side friction is projected to be zero. It is not known if drivers will actually desire zero side friction at those speeds.

Running Speed for Curves with High Design Speeds

In addition to the side friction, Method 5 requires a running speed for horizontal curves. A curve radius is then found using this running speed along with zero side friction. The running speeds shown in *Green Book* Exhibit 3-14 date back to at least the 1965 *Blue Book*. At that time, it was assumed that vehicles on a roadway would be operated at a speed that was lower than the design speed. This assumption permeated the design guidance in the 1965 *Blue Book* as well as later guidance. In 2001, running speed was removed from stopping sight distance calculations because research and experience indicated that drivers tended to operate their vehicles at or even above the design speed of the roadway. At that time, running speed was officially removed from the *Green Book*, so the explanation of its origin was also removed. However, there are several design subjects within the *Green Book* that still use running speed. Horizontal curve design is one of those subjects. Due to lack of information about driver behavior at high speeds, the running speed was extrapolated based on the existing distribution. The resulting values are shown in Figure 8-9.

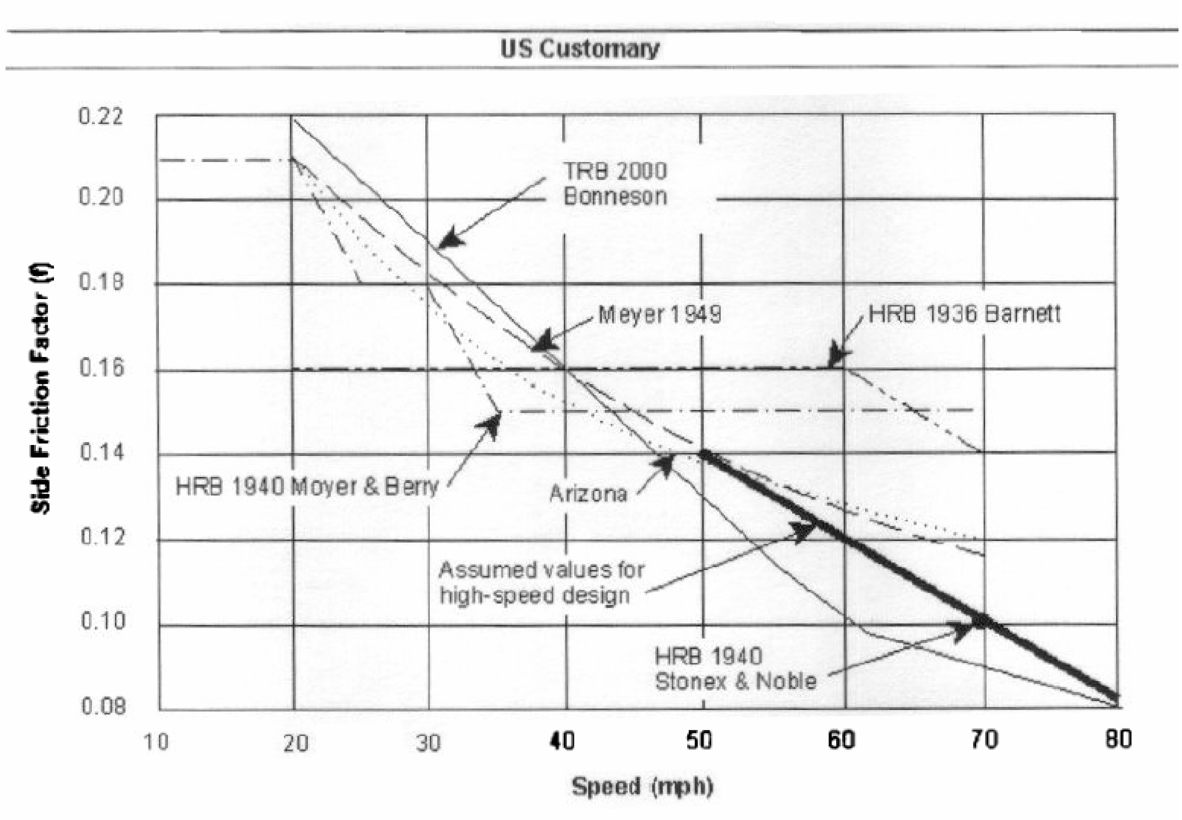
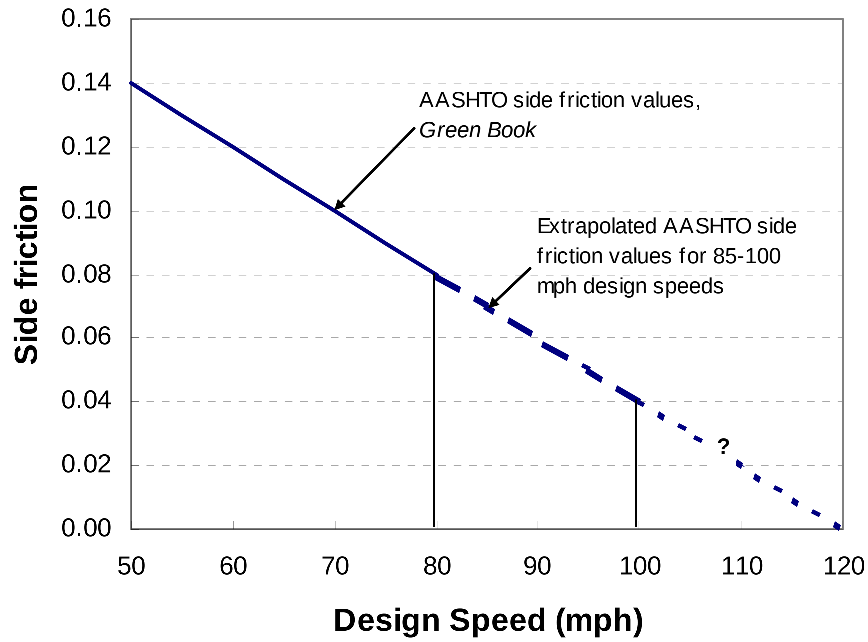
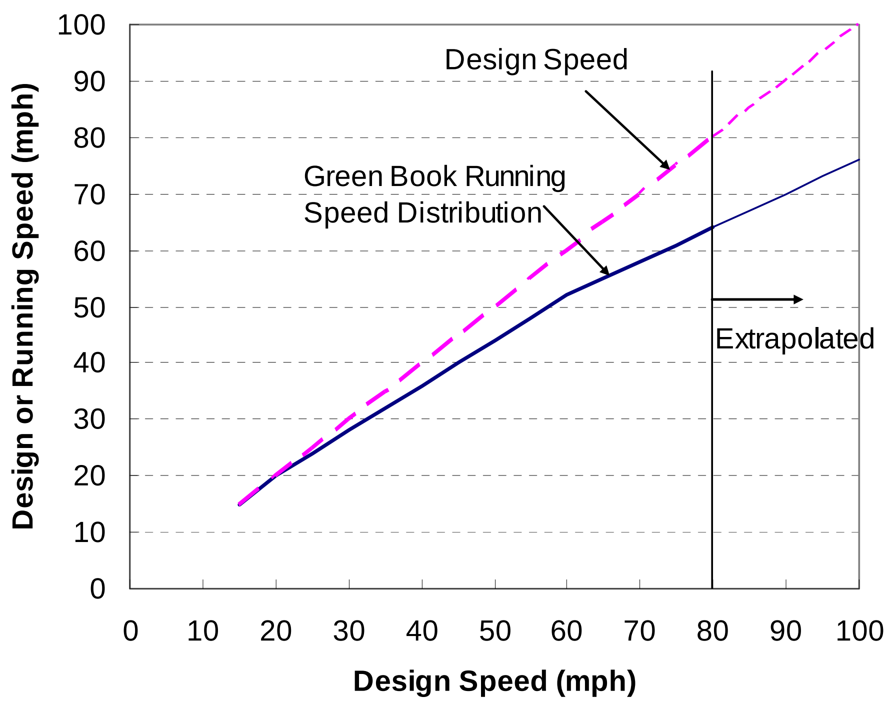


Figure 8-7. Side Friction Factors for High-Speed Streets and Highways from *Green Book* (GB Exhibit 3-10) (*2*).



Note: 1 mph = 1.61 km/h

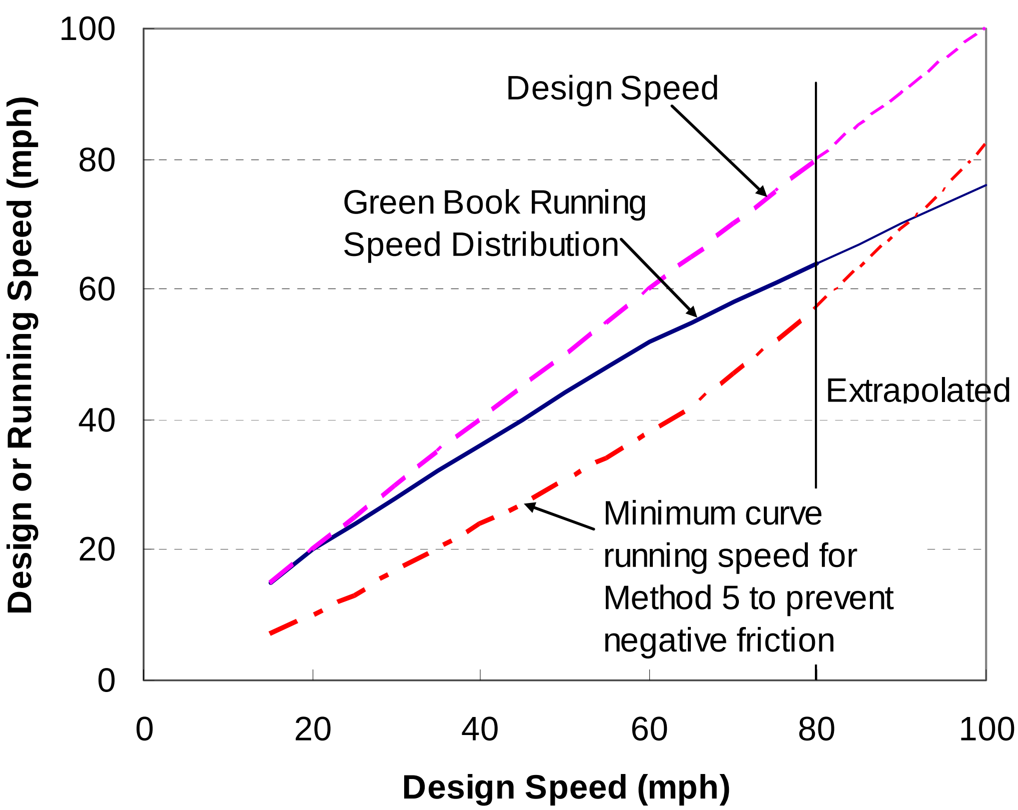
Figure 8-8. Potential Side Friction Factors for High Design Speeds (Extrapolated from *Green Book*).



Note: 1 mph = 1.61 km/h

Figure 8-9. Potential Running Speeds for Curves (Extrapolated from *Green Book*).

During the application of Method 5 to high design speeds, some anomalous results occurred at the highest speeds. Method 5 was suggesting curve radii that were *smaller* than the calculated minimum curve radius. After some investigation, it was found that the extrapolation of the running speeds created these anomalies. The running speed is used in Method 5 to determine a curve radius at which a vehicle's lateral force is completely compensated for by superelevation alone (i.e., it uses no side friction). At 95 mph [153 km/h] and 100 mph [161 km/h], the extrapolation of the running speed distribution was lower than the minimum speed on the curve that required side friction (see Figure 8-10). Vehicles at the extrapolated *Green Book* running speeds were actually generating "negative friction"; that is, those vehicles were tending to slide toward the *inside* of the curve instead of the outside. This situation is undesirable because it is contrary to driver expectations. It also causes Method 5 to produce anomalous curve radii. To prevent this situation, an additional constraint was added to the running speed assumptions. The running speed was assumed to be either the *Green Book* extrapolation or the speed to prevent negative friction, whichever was greater. The additional constraint prevents the anomalies from occurring.

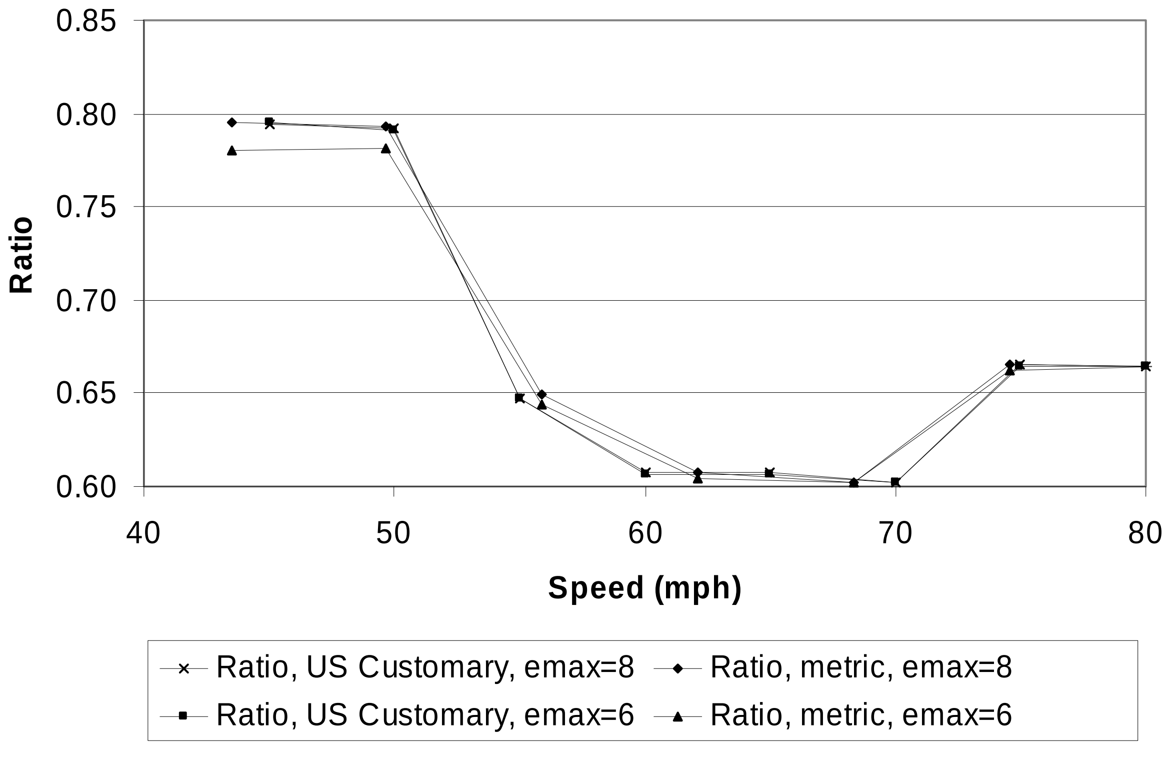


Note: 1 mph = 1.61 km/h

Figure 8-10. Running Speeds for Curves versus Speeds to Prevent Negative Friction.

TxDOT Minimum Curve Radii

TRDM uses a modified approach for minimum curve radii compared to the *Green Book*. TxDOT developed its own curvature standards before the AASHTO standards appeared, and when these curvature standards were developed, TxDOT used a lower value for available side friction than AASHTO eventually selected. Therefore, Texas' curve radii were substantially larger than the "minimum radii" in AASHTO's guidance. Currently, TRDM shows its original, larger recommended minimum curve radii as a "usual" minimum to be used in new designs, with the *Green Book* values presented as an "absolute" minimum, to be used only where unusual design circumstances dictate. The relationship between the "usual" and "absolute" minimum radii is not provided in TRDM. The ratio between "usual" and "absolute" was calculated and a plot of the values is shown in Figure 8-11. The ratios vary with speed, although a logical trend for the variation is not obvious. Therefore, to extrapolate to higher speeds, the "usual" minimum radius for curvature was set using the same proportions as for the 75-mph [120 km/h] and 80-mph [128 km/h] design speeds, which was determined as being 0.66. Because the "usual" minimum radius is larger than the "absolute" minimum radius, a curve of the "usual" minimum radius does not use the maximum amount of available superelevation.



Note: 1 mph = 1.61 km/h

Figure 8-11. Calculated Ratio of "Absolute" Minimum to "Usual" Minimum.

Speed Reduction on Curves

One other important design consideration is the potential for drivers to slow down on highway curves. Bonneson (47) developed a regression model for vehicle speed on curves as part of NCHRP Report 439. His model is shown in Figure 8-12. He found that vehicles tended to reduce their speed on curves, with the speed reduction increasing as the radius of the curve approached the *Green Book*'s minimum radius for design speeds up to 70 mph [112 km/h].

Note that the superelevation rate used in this model is the actual superelevation rate of the curve in question, not the maximum superelevation. For curves with radius R_{min}, the superelevation rate used to estimate the speed reduction will be e_{max}. For curves with radii larger than R_{min}, the superelevation rate used will be less than e_{max}. Also note that the model uses metric values only to calculate the vehicle speed on a curve, and the result must be converted to US customary units.

For a passenger car traveling at the design speed (assumed to be the 85th percentile speed of the roadway):

$$V_s = V_D - V_c$$

Where:

V_s = speed reduction on a curve (mph or km/h),
V_D = design speed of curve (mph or km/h), and
V_c = calculated vehicle speed on curve (mph or km/h).

$$V_c = \frac{63.5R\left(-0.0133 + \sqrt{0.0133^2 + \dfrac{4c}{127R}}\right)}{w}$$

Where:

R = minimum curve radius for the design speed (meters) and
w = 1.0 if metric, 1.6 if US customary.

$$c = e + 0.256 + \left(0.0133 - \left(0.00223\right)\right)V_D$$

Where:

e = superelevation rate of the curve and
V_D = design speed of curve in km/h.

Figure 8-12. Bonneson's Model for Speed Reduction on Curves (47).

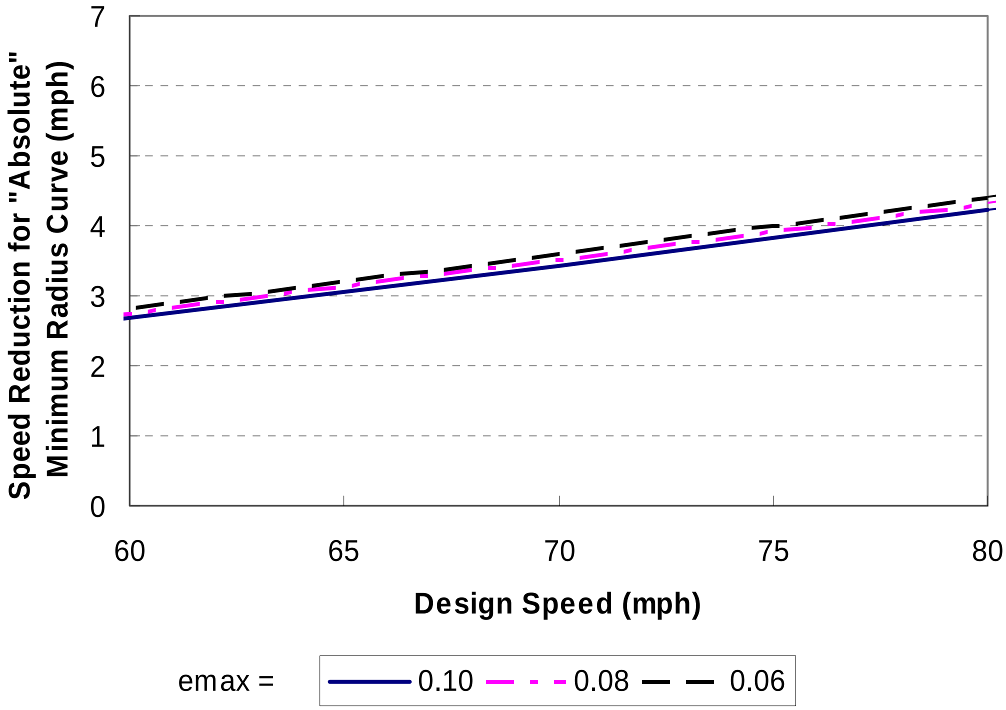
Bonneson also developed models for trucks. The results of the truck model indicate that trucks will have approximately 2 to 2.5 mph [3 to 4 km/h] additional reduction in speed compared to passenger cars for the same curve, so the passenger car results plus an additional 2 mph [3 km/h] can be used for trucks.

Figure 8-13 shows the model estimates of speed reductions for superelevation rates of 6 percent, 8 percent, and 10 percent. The radii used in Figure 8-13 are the "absolute" minimum curve radii from the *Green Book*, and include only design speeds up to 80 mph [128 km/h]. There is very little difference in the speed reductions between the various superelevation rates at these speeds, with approximately a 4 mph speed reduction at 80 mph [128 km/h].

Figure 8-14 shows calculated speed reductions for "absolute" minimum curve radii for design speeds above 80 mph [128 km/h]. For 6 percent maximum superelevation, the speed reduction for the "absolute" minimum radius curve could reach 6 mph [9.7 km/h] at 100-mph [161 km/h] design speed. An important observation from Figures 8-14 and 8-15 is that a speed reduction is predicted for all curve designs using minimum radius for design speeds above 60 mph [96 km/h].

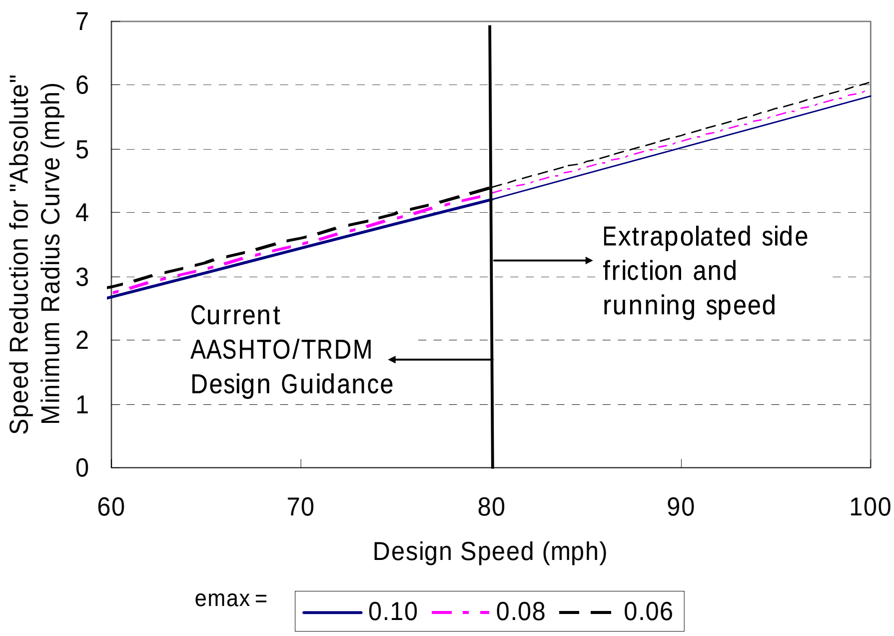
At TxDOT's "usual" minimum radius, the curve speed reduction should be somewhat less because the radius of the curve is larger. Figure 8-15 illustrates the speed reduction for "usual" minimum curve radii. The model's behavior in this instance is complex due to the interrelated changes in curve radius and superelevation that occur at the "usual" minimum radius.
Figure 8-17 illustrates that speed reductions will occur not only at the minimum curve radius, but at larger radii as well.

All of the information presented so far has been presented for either the "absolute" minimum curve radius or TxDOT's "usual" minimum curve radius. Theoretically, a highway curve could be designed to provide zero speed reduction. However, Bonneson's model returns infinite curve radii (i.e., a tangent) for design speeds as low as 80 mph [128 km/h] when attempting to find the curve radii with zero speed reduction. In fact, the model predicts a speed reduction on a normal crown of 2 percent for speeds of 80 mph [128 km/h], which implies that speed reduction would occur even on a tangent with a normal cross slope. The model is likely producing unreliable results in this instance. **Further research is needed to determine whether or not drivers will slow down on horizontal curves on high-speed roadways, and if they do, by how much.**



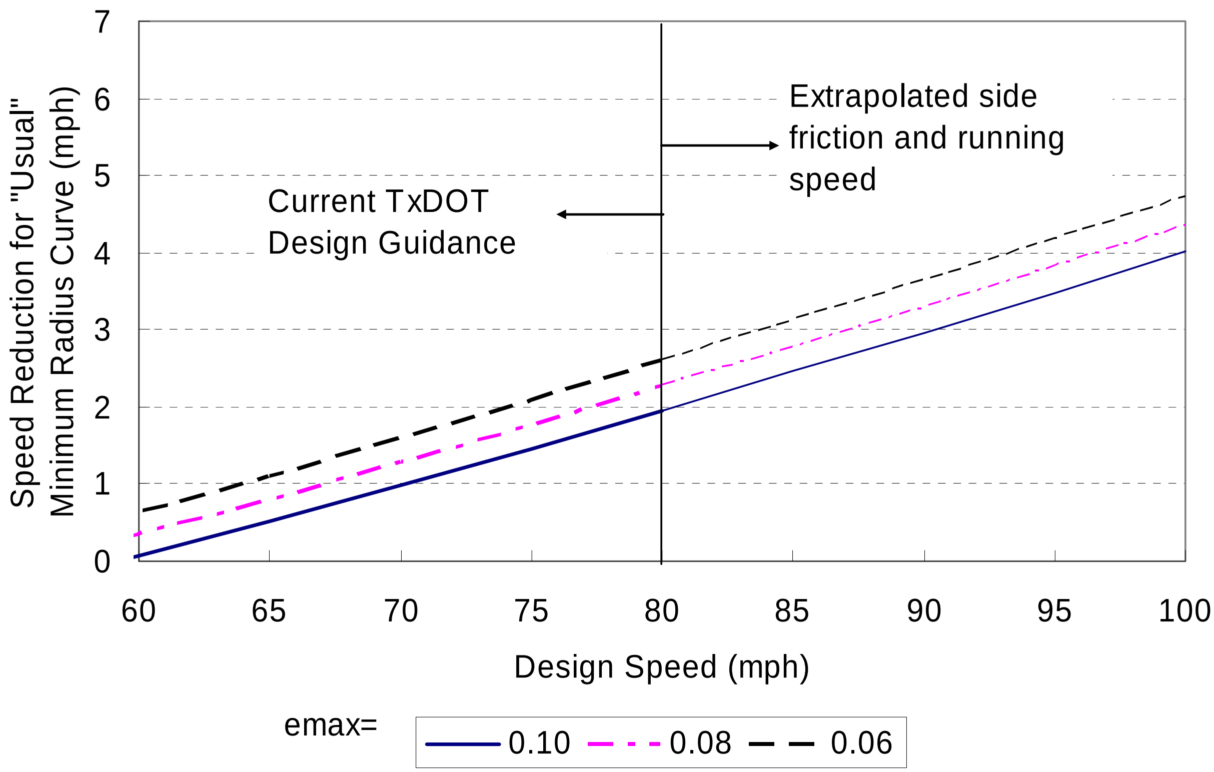
Note: 1 mph = 1.61 km/h

Figure 8-13. Speed Reductions for Passenger Cars for "Absolute" Minimum Curve Radii.



Note: 1 mph = 1.61 km/h

Figure 8-14. Speed Reductions for Passenger Cars for "Absolute" Minimum Curve Radii at High Design Speeds.



Note: 1 mph = 1.61 km/h

Figure 8-15. Speed Reductions for Passenger Cars for "Usual" Minimum Curve Radii.

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

Superelevation

The maximum superelevation rates of 6 to 8 percent are set for environmental reasons and are not varied based on design speed. However, the design radii for curves with a 6 percent maximum superelevation are very large for high design speeds. Because icing conditions are rare in many parts of Texas, it may be reasonable to use maximum superelevation rates greater than 8 percent for roadways with high design speeds. **Further research into the potential use of higher superelevation rates is needed, particularly in relation to climatic conditions in Texas and the use of modern pavement types, such as porous friction courses.**

Maximum Side Friction

Potential maximum side friction values for design speeds above 80 mph [128 km/h] were presented in Figure 8-8. These values are also shown in Table 8-3. **The linear extrapolation may or may not be reasonable at these speeds, and further research is necessary to determine the shape of the side friction distribution at design speeds above 80 mph [128 km/h].**

Running Speed

The running speed distribution, corrected to eliminate negative friction, was shown in Figure 8-9. The values are also included in Table 8-3, for both US customary and metric units. **The use of a speed in determining horizontal curvature that is less than the design speed deserves additional investigation, especially for the high-design-speed condition.**

Table 8-3. Side Friction Factors and Running Speeds for Horizontal Curves.

(US Customary)			(Metric)		
Design Speed (mph)	**Side Friction Factor**	**Running Speed (mph)**	**Design Speed (km/h)**	**Side Friction Factor**	**Running Speed (km/h)**
15	0.32	15	20	0.35	20
20	0.27	20	30	0.28	30
25	0.23	24	40	0.23	40
30	0.20	28	50	0.19	47
35	0.18	32	60	0.17	55
40	0.16	36	70	0.15	63
45	0.15	40	80	0.14	70
50	0.14	44	90	0.13	77
55	0.13	48	100	0.12	85
60	0.12	52	110	0.11	91
65	0.11	55	120	0.09	98
70	0.10	58	130	0.08	102
75	0.09	61	140	0.07	110
80	0.08	64	150	0.05	118 [a]
85	0.07	67	160	0.04	131 [a]
90	0.06	70			
95	0.05	75 [a]			
100	0.04	82 [a]			

[a] Values adjusted to eliminate negative friction on curve.
Shaded areas reflect high-design-speed potential values.

Horizontal Curvature

The minimum curve radii for superelevation rates of 6 percent and 8 percent are shown in Table 8-4. These radii were calculated using the horizontal curve equation, with the side friction values in Table 8-3 and the assumed maximum superelevation rates. In addition, the amount of expected speed reduction for these curve radii, for both passenger cars and trucks, is also shown in the tables.

Minimum Curve Radii without Additional Superelevation

TRDM calculates the minimum curve radius without superelevation by stating that the normal 2 percent crown is maintained through the curve, and that the e_{max} used was 8 percent (*1*). Maintaining a normal crown would result in a -2 percent superelevation for one direction. If the side friction is not excessive for that direction, then a normal crown can be used. Otherwise, a reverse crown would be used. Use of negative superelevation is contrary to driver expectations, however, and might not be appropriate at high speeds. **Further research is needed to determine whether negative superelevation can be comfortably used at these speeds.**

Table 8-5 shows the minimum curve radii without additional superelevation and an e_{max} of 8 percent, consistent with the definition in TRDM Table 2-4.

Superelevation Rates for Curves with Radii Greater than R_{min}

TxDOT is currently revising the TRDM tables relating superelevation to various curve radii to conform to the current *Green Book* guidance for side friction factors. Tables 8-6 through 8-9 show curve radii for maximum superelevation rates of 6 percent and 8 percent in the preferred format of the TRDM. These tables use the side friction and running speed distributions shown in Table 8-3.

Superelevation Transition

Table 8-10 shows potential maximum relative gradients for superelevation transition for all design speeds. The values for the highest design speeds were extrapolated from the rates for lower speeds. The 0.025 percent reduction for each 5 mph [8 km/h] increase in design speed was extended to design speeds above 80 mph [128 km/h]. The corresponding metric values were also generated this way. Special attention should be paid to roadway drainage during superelevation transition to prevent ponding, especially in conjunction with vertical curvature.

Table 8-4. Potential Minimum Curve Radius and Speed Reduction.

(US Customary)				
Design Speed (mph)	**"Usual" minimum radius of curvature [a] (ft)**	**Speed Reduction, cars/trucks [b] (mph)**	**Absolute minimum radius of curvature [c] (ft)**	**Speed Reduction, cars/trucks [b] (mph)**
Minimum Radius for e_{max} of 6 percent				
75	3775	2.1/4.1	2510	4.0/6.0
80	4605	2.6/4.6	3060	4.4/6.4
85	5615	3.1/5.1	3710	4.8/6.8
90	6820	3.6/5.6	4500	5.2/7.2
95	8285	4.2/6.2	5470	5.6/7.6
100	10100	4.7/6.7	6670	6.0/8.0
Minimum Radius for e_{max} of 8 percent				
75	3330	1.8/3.8	2215	3.9/5.9
80	4025	2.2/4.2	2675	4.3/6.3
85	4865	2.8/4.8	3215	4.7/6.7
90	5845	3.3/5.3	3860	5.1/7.1
95	7010	3.8/5.8	4630	5.5/7.5
100	8420	4.4/6.4	5560	5.9/7.9

(Metric)				
Design Speed (km/h)	**"Usual" minimum radius of curvature [a] (m)**	**Speed Reduction, cars/trucks [b] (km/h)**	**Absolute minimum radius of curvature [c] (m)**	**Speed Reduction, cars/trucks [b] (km/h)**
Minimum Radius for e_{max} of 6 percent:				
120	1140	3.5/6.5	755	6.5/9.5
130	1430	4.6/7.6	950	7.4/10.4
140	1800	5.7/8.7	1190	8.4/11.4
150	2440	6.5/9.5	1615	8.7/11.7
160	3050	7.6/10.6	2020	9.7/12.7
Minimum Radius for e_{max} of 8 percent:				
120	1000	3.0/6.0	665	6.3/9.3
130	1250	4.1/7.1	830	7.3/10.3
140	1560	5.2/8.2	1030	8.2/11.2
150	2060	5.9/8.9	1365	8.6/11.6
160	2550	7.1/10.1	1680	9.5/12.9

[a] Methodology used to determine the "usual" minimum values included in the TRDM was not identified. The ratio of absolute minimum to usual minimum for TRDM values for 75 and 80 mph design speeds were calculated as 0.66. The "usual" minimum values for design speeds of 85 to 100 mph were calculated as absolute minimum/0.66.

[b] Determined using equation in Figure 8-12.

[c] Calculated using $R_{\min} = \dfrac{V_D^2}{C\left(e_{\max} + f_{\max}\right)}$,

with R_{min} = absolute minimum curve radius (ft or m), V_D = design speed (mph or km/h), C = conversion constant (15 for US customary units, 127 for metric), e_{max} = maximum curve superelevation, and f_{max} = maximum available side friction factor (Table 8-3).

Shaded areas reflect high-design-speed potential values.

Table 8-5. Potential Minimum Curve Radius without Additional Superelevation.

(US Customary)	
Design Speed (mph)	**Minimum Radius (ft)**
70	10750
75	12000
80	13340
85	14700
90	16200
95	18800
100	22400
(Metric)	
Design Speed (km/h)	**Minimum Radius (m)**
120	3650
130	4015
140	4680
150	5480
160	6750
Shaded areas reflect high-design-speed potential values.	

Table 8-6. Potential Superelevation Rates for Curves with 6 Percent Maximum Superelevation (US Customary).

Superelevation Rates for Horizontal Curves on High-Speed Highways Superelevation Rate, e (6%), for Design Speed of: (US Customary)							
Radius (ft)	**70 mph**	**75 mph**	**80 mph**	**85 mph**	**90 mph**	**95 mph**	**100 mph**
23000	NC	NC	NC	NC	NC	NC	NC
20000	NC	NC	NC	NC	NC	NC	2.2
17000	NC	NC	NC	NC	NC	2.2	2.6
14000	NC	NC	NC	RC	2.3	2.6	3.2
12000	NC	NC	2.1	2.4	2.6	3.0	3.6
10000	2.1	2.3	2.5	2.8	3.1	3.6	4.3
8000	2.5	2.8	3.1	3.4	3.8	4.5	5.3
6000	3.2	3.6	4.0	4.5	5.0	5.8	R_{min} = 6670 ft
5000	3.7	4.2	4.7	5.2	5.8	R_{min} = 5470 ft	
4000	4.4	4.9	5.5	5.9	R_{min} = 4500 ft		
3500	4.9	5.4	5.9	R_{min} = 3710 ft			
3000	5.3	5.8	R_{min} = 3060 ft				
2500	5.8	R_{min} = 2510 ft					
2000	R_{min} = 2050 ft						
NC = Normal Crown, RC = Reverse Crown, e_{max} = 6% Shaded areas reflect high-design-speed potential values.							

Table 8-7. Potential Superelevation Rates for Curves with 8 Percent Maximum Superelevation (US Customary).

Superelevation Rates for Horizontal Curves on High-Speed Highways Superelevation Rate, e (8%), for Design Speed of: (US Customary)							
Radius (ft)	**70 mph**	**75 mph**	**80 mph**	**85 mph**	**90 mph**	**95 mph**	**100 mph**
23000	NC	NC	NC	NC	NC	NC	NC
20000	NC	NC	NC	NC	NC	NC	2.2
17000	NC	NC	NC	NC	NC	2.2	2.6
14000	NC	NC	NC	2.1	2.3	2.7	3.2
12000	NC	NC	2.2	2.4	2.7	3.1	3.7
10000	2.1	2.4	2.6	2.9	3.2	3.7	4.5
8000	2.6	2.9	3.3	3.6	4.0	4.7	5.6
6000	3.4	3.8	4.3	4.8	5.3	6.2	7.4
5000	4.1	4.5	5.1	5.7	6.4	7.5	R_{min} = 5560 ft
4000	4.9	5.5	6.2	7.0	7.9	R_{min} = 4630 ft	
3500	5.5	6.2	7.0	7.8	R_{min} = 3860 ft		
3000	6.3	7.0	7.8	R_{min} = 3215 ft			
2500	7.2	7.8	R_{min} = 2675 ft				
2000	7.9	R_{min} = 2215 ft					
1800	R_{min} = 1820 ft						

NC = Normal Crown, e_{max} = 8%
Shaded areas reflect high-design-speed potential values.

Table 8-8. Potential Superelevation Rates for Curves with 6 Percent Maximum Superelevation (Metric).

Superelevation Rates for Horizontal Curves on High-Speed Highways Superelevation Rate, e (6%), for Design Speed of: (Metric)						
Radius (m)	**110 km/h**	**120 km/h**	**130 km/h**	**140 km/h**	**150 km/h**	**160 km/h**
7000	NC	NC	NC	NC	NC	NC
5000	NC	NC	NC	NC	2.1	2.6
3000	NC	2.3	2.5	3.0	3.5	4.3
2500	2.3	2.7	3.0	3.5	4.2	5.1
2000	2.8	3.3	3.7	4.3	5.2	R_{min} = 2015 m
1500	3.6	4.2	4.7	5.5	R_{min} = 1610 m	
1400	3.8	4.4	5.0	5.7		
1300	4.0	4.7	5.3	5.9		
1200	4.2	5.0	5.6	6.0		
1000	4.8	5.6	6.0	R_{min} = 1190 m		
900	5.1	5.8	R_{min} = 950 m			
800	5.4	6.0				
700	5.8	R_{min} = 755 m				
600	6.0					
500	R_{min} = 560 m					
NC = Normal Crown, e_{max} = 6% Shaded areas reflect high-design-speed potential values.						

Table 8-9. Potential Superelevation Rates for Curves with 8 Percent Maximum Superelevation (Metric).

Superelevation Rates for Horizontal Curves on High-Speed Highways Superelevation Rate, e (8%), for Design Speed of: (Metric)						
Radius (m)	**110 km/h**	**120 km/h**	**130 km/h**	**140 km/h**	**150 km/h**	**160 km/h**
7000	NC	NC	NC	NC	NC	NC
5000	NC	NC	NC	NC	2.2	2.7
3000	2.1	2.4	2.6	3.1	3.6	4.5
2500	2.4	2.9	3.1	3.7	4.4	5.4
2000	3.0	3.5	3.9	4.6	5.5	6.7
1500	3.9	4.6	5.1	6.0	7.3	R_{min} = 1680 m
1400	4.1	4.9	5.4	6.4	7.8	
1300	4.4	5.2	5.8	6.9	R_{min} = 1365 m	
1200	4.7	5.6	6.3	7.4		
1000	5.5	6.5	7.4	R_{min} = 1030 m		
900	6.0	7.1	7.9			
800	6.6	7.6	R_{min} = 830 m			
700	7.2	8.0				
600	7.7	R_{min} = 665 m				
500	8.0					
400	R_{min} = 500 m					
NC = Normal Crown, e_{max} = 8% Shaded areas reflect high-design-speed potential values.						

Table 8-10. Potential Maximum Relative Gradients for Superelevation Transition.

(US Customary)			(Metric)		
Design Speed (mph)	**Maximum Relative Gradient (%)[a]**	**Equivalent Maximum Relative Slope**	**Design Speed (km/h)**	**Maximum Relative Gradient (%)[a]**	**Equivalent Maximum Relative Slope**
15	0.78	1:128	20	0.80	1:125
20	0.74	1:135	30	0.75	1:133
25	0.70	1:143	40	0.70	1:143
30	0.66	1:152	50	0.65	1:150
35	0.62	1:161	60	0.60	1:167
40	0.58	1:172	70	0.55	1:182
45	0.54	1:185	80	0.50	1:200
50	0.50	1:200	90	0.47	1:213
55	0.47	1:213	100	0.44	1:227
60	0.45	1:222	110	0.41	1:244
65	0.43	1:233	120	0.38	1:263
70	0.40	1:250	130	0.35	1:286
75	0.38	1:263	140	0.32	1:313
80	0.35	1:286	150	0.28	1:357
85	0.33	1:303	160	0.25	1:400
90	0.30	1:333			
95	0.28	1:357			
100	0.25	1:400			

[a] Maximum relative gradient for profile between edge of traveled way and axis of rotation.
Shaded areas reflect high-design-speed potential values.

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- **Further research into side friction factors is needed to determine whether the assumed side friction distribution is correct for speeds above 80 mph [128 km/h].** The existing studies in this area are either out of date (i.e., based on outdated vehicles, tire designs, and driver performance characteristics) or do not encompass the necessary vehicle speeds, or both.
- **The use of a speed in determining horizontal curvature that is less than the design speed deserves additional investigation, especially for the high-design-speed condition.** The use of "running speed" in Method 5 may or may not be reasonable at high speeds.
- Drivers expect superelevation when entering a curve at most speeds. However, negative superelevation may be uncomfortable for drivers, depending on the speed and radius of the particular curve. **Further research in this area is needed, especially in relation to high design speeds.**
- Currently, vehicle performance data are lacking on superelevation transition sections for design speeds above 80 mph [128 km/h]. Different design assumptions may apply at these speeds than is currently known, which may require a different superelevation transition design. In addition, truck performance may be affected by the length of the transition, especially if the truck has a high center of gravity. **Research in this area is needed to determine whether current transition techniques are adequate for both driver safety and driver comfort at high speeds.**
- **Data on driver behavior and vehicle capabilities for design speeds above 80 mph [128 km/h] are lacking.** It is possible that some vehicle types may be more affected by horizontal curvature than others, especially if the vehicle has a high center of gravity. More research is needed in this area.
- The design radii for curves with a 6 percent maximum superelevation are very large for high design speeds, exceeding 10,000 ft [3050 m] for 100 mph [161 km/h]. Because icing conditions are rare in many parts of Texas, it may be reasonable to use maximum superelevation rates greater than 8 percent for roadways with high design speeds. **Further research into the potential use of higher superelevation rates is needed, particularly in relation to climatic conditions in Texas and the use of modern pavement types, such as porous friction courses.**

CHAPTER 9

RAMP DESIGN SPEED

CURRENT GUIDANCE

The TxDOT *Roadway Design Manual* states that "there should be a definite relationship between the design speed on a ramp or direct connection and the design speed on the intersecting highway or frontage road" (1). The TRDM includes a table that "shows guide values for ramp/connection design speed." The table includes ramp design speeds for upper range (85 percent), mid range (70 percent) and lower range (50 percent) categories. The TRDM notes that the AASHTO's *A Policy on Geometric Design of Highways and Streets* (commonly known as the *Green Book*) (*2*) provides additional guidance on the application of the ranges of ramp design speed.

OTHER GUIDANCE

Minimum design criteria and guidelines were developed for TxDOT for use on the TTC-35 high-priority corridor (*11*). Their recommendations for direct connectors and ramps for mainlanes design speed of 80 mph [129 km/h] are listed in Table 9-1

Table 9-1. TTC-35 Design Criteria (Main Lane Design Speed is 80 mph [129 km/h]).

	Passenger Car Facility	Truck Facility
Direct Connectors	• 70 mph [113 km/h] (desirable) • 60 mph [97 km/h] (usual)	• 80 mph [129 km/h] (desirable) • 70 mph [113 km/h] (usual)
Ramps[a]	• 80 mph [129 km/h]	• 70 mph [113 km/h] (main lane) • 60 mph [97 km/h] (mid ramp) • 50 mph [81 km/h] (FR/AR)

[a] Ramps between the passenger car facility and truck facility are passenger car ramps. For criteria for ramps between other facilities and truck facilities, see truck facility ramps. For truck facility ramps, the mainlane area is that portion of the ramp adjacent to the mainlanes and that area within 300 ft [92 m] of the actual mainlane gore. The frontage road/access road (FR/AR) area is that area adjacent to the frontage road or access road and that area within 300 ft [92 m] of the actual FR/AR gore or within 600 ft [183 m] of intersecting road if there is no FR/AR. The mid ramp area is that area between the mainlane area and the FR/AR area.

DISCUSSION

Ramp design speeds generally (but not always) represent increments rounded to nearest 5-mph increment (or 10 km/h) of 85, 70, or 50 percent of the highway design speed. Table 9-2 lists the current values included in the TRDM. In addition, the table provides the value calculated using the 85, 70, or 50 percent factor along with the difference between the calculated value and the value included in the TRDM (shown as bold values). Figure 9-1 shows the calculated and rounded values in a plot.

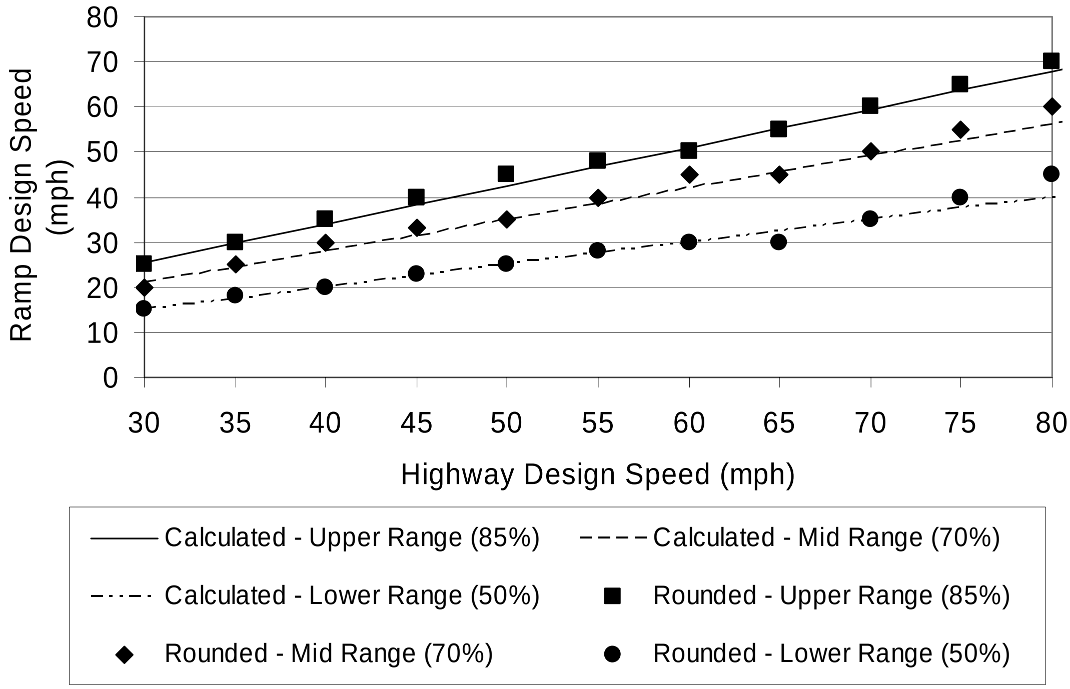
Table 9-2. Guide Values for Ramp/Connection Design Speed as Related to Highway Design Speed (Rounded Values are from TRDM Table 3-20).

(US Customary)											
Highway Design Speed (mph)	**30**	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**	**75**	**80**
Calculated-Upper Range (85%)	26	30	34	38	43	47	51	55	60	64	68
Calculated-Mid Range (70%)	21	25	28	32	35	39	42	46	49	53	56
Calculated-Lower Range (50%)	15	18	20	23	25	28	30	33	35	38	40
Rounded-Upper Range (85%)	**25**	**30**	**35**	**40**	**45**	**48**	**50**	**55**	**60**	**65**	**70**
Rounded-Mid Range (70%)	**20**	**25**	**30**	**33**	**35**	**40**	**45**	**45**	**50**	**55**	**60**
Rounded-Lower Range (50%)	**15**	**18**	**20**	**23**	**25**	**28**	**30**	**30**	**35**	**40**	**45**
Difference-Upper Range (85%)	-1	0	1	2	3	1	-1	0	1	1	2
Difference-Mid Range (70%)	-1	1	2	2	0	2	3	-1	1	3	4
Difference-Lower Range (50%)	0	1	0	1	0	1	0	-3	0	3	5

(Metric)									
Highway Design Speed (km/h)	**50**	**60**	**70**	**80**	**90**	**100**	**110**	**120**	**130**
Calculated-Upper Range (85%)	43	51	60	68	77	85	94	102	111
Calculated-Mid Range (70%)	35	42	49	56	63	70	77	84	91
Calculated-Lower Range (50%)	25	30	35	40	45	50	55	60	65
Rounded-Upper Range (85%)	**40**	**50**	**60**	**70**	**80**	**90**	**100**	**110**	**120**
Rounded-Mid Range (70%)	**30**	**40**	**50**	**60**	**60**	**70**	**80**	**90**	**100**
Rounded-Lower Range (50%)	**20**	**30**	**40**	**40**	**50**	**50**	**60**	**70**	**80**
Difference-Upper Range (85%)	-3	-1	1	2	4	5	7	8	10
Difference-Mid Range (70%)	-5	-2	1	4	-3	0	3	6	9
Difference-Lower Range (50%)	-5	0	5	0	5	0	5	10	15

Bold values = guide values included in TRDM Table 3-20.

As an example, for a ramp from a freeway with a design speed of 75 mph [121 km/h], the ramp design speed (see Table 9-2) would be 65 mph [105 km/h] if the upper range (85 percent) is selected. The ramp design speed that would have been calculated if the 85 percent factor was used is 64 mph [103 km/h]; therefore, the value in TRDM has a 1-mph [1.6 km/h] increase as a result of the rounding. The amount of rounding, either down or up, between the values calculated assuming a 85, 70, or 50 percent factor and the TRDM values can be used to provide guidance in generating ramp design speeds for the higher speed facilities. In all but one case, the rounding was either within 1 mph [1.6 km/h] or represented a higher speed. The exception was for a 65-mph [105 km/h] highway design speed and the lower range condition. The calculated value is 33 mph [53 km/h] and the value in the TRDM is 30 mph [48 km/h] – a 3-mph [5 km/h] decrease. Inspecting the metric values gives a different view. Several of the rounded values were more than 5 mph [8 km/h] and in one case was 9 mph [15 km/h] different from the calculated value.



Note: 1 mph = 1.61 km/h

Figure 9-1. Plot of Calculated and Rounded Ramp Design Speeds.

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

Similar to facilities with design speeds of 80 mph [129 km/h] or less, ramps on high-design-speed facilities should also have a relationship between the ramp design speed and the mainlane design speed. The current relationship, in general, is for the ramp design speed to be 85, 70, or 50 percent of the highway design speed, rounded up to the nearest 5-mph [8 km/h] increment. If this relationship is used for the high design speeds, then the ramp design speeds as shown in Table 9-3 and Figure 9-2 would be generated.

Table 9-3. Guide Values for Ramp/Connection Design Speed as Related to Highway Design Speed for Highway Speeds up to 100 mph [160 km/h] (Rounded Values for 30 to 80 mph [50 to 130 km/h] are from TRDM Table 3-20).

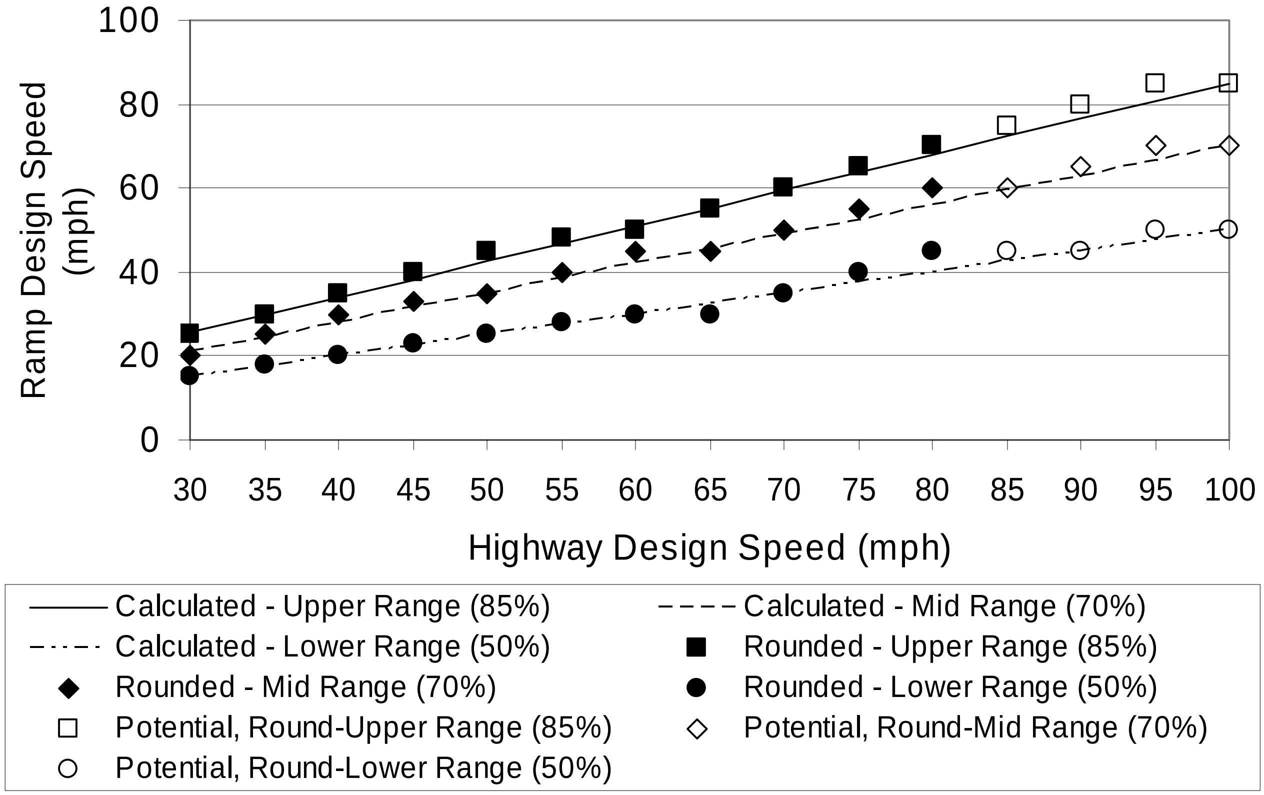
(US Customary)

Source-Range	**Ramp Design Speed (mph)**														
	For Existing Highway Design Speed (mph)											**For Potential**			
	30	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**	**75**	**80**	**85**	**90**	**95**	**100**
Calculated-Upper Range (85%)	26	30	34	38	43	47	51	55	60	64	68	72	77	81	85
Calculated-Mid Range (70%)	21	25	28	32	35	39	42	46	49	53	56	60	63	67	70
Calculated-Lower Range (50%)	15	18	20	23	25	28	30	33	35	38	40	43	45	48	50
Rounded (Manual)-Upper Range (85%)	**25**	**30**	**35**	**40**	**45**	**48**	**50**	**55**	**60**	**65**	**70**	75	80	85	85
Rounded (Manual)-Mid Range (70%)	**20**	**25**	**30**	**33**	**35**	**40**	**45**	**45**	**50**	**55**	**60**	60	65	70	70
Rounded (Manual)-Lower Range (50%)	**15**	**18**	**20**	**23**	**25**	**28**	**30**	**30**	**35**	**40**	**45**	45	45	50	50
Difference-Upper Range (85%)	-1	0	1	2	3	1	-1	0	1	1	2	3	4	4	0
Difference-Mid Range (70%)	-1	1	2	2	0	2	3	-1	1	3	4	1	2	4	0
Difference-Lower Range (50%)	0	1	0	1	0	1	0	-3	0	3	5	3	0	3	0

(Metric)

Source-Range	**Ramp Design Speed (km/h)**											
	For Existing Highway Design Speed (km/h)									**For Potential**		
	50	**60**	**70**	**80**	**90**	**100**	**110**	**120**	**130**	**140**	**150**	**160**
Calculated-Upper Range (85%)	43	51	60	68	77	85	94	102	111	119	128	136
Calculated-Mid Range (70%)	35	42	49	56	63	70	77	84	91	98	105	112
Calculated-Lower Range (50%)	25	30	35	40	45	50	55	60	65	70	75	80
Rounded (Manual)-Upper Range (85%)	**40**	**50**	**60**	**70**	**80**	**90**	**100**	**110**	**120**	120	130	140
Rounded (Manual)-Mid Range (70%)	**30**	**40**	**50**	**60**	**60**	**70**	**80**	**90**	**100**	100	110	120
Rounded (Manual)-Lower Range (50%)	**20**	**30**	**40**	**40**	**50**	**50**	**60**	**70**	**80**	70	80	80
Difference-Upper Range (85%)	-3	-1	1	2	4	5	7	8	9	1	3	4
Difference-Mid Range (70%)	-5	-2	1	4	-3	0	3	6	9	2	5	8
Difference-Lower Range (50%)	-5	0	5	0	5	0	5	10	15	0	5	0

Bold values = guide values included in TRDM Table 3-20.
Shaded areas reflect high-design-speed potential values.



Note: 1 mph = 1.61 km/h

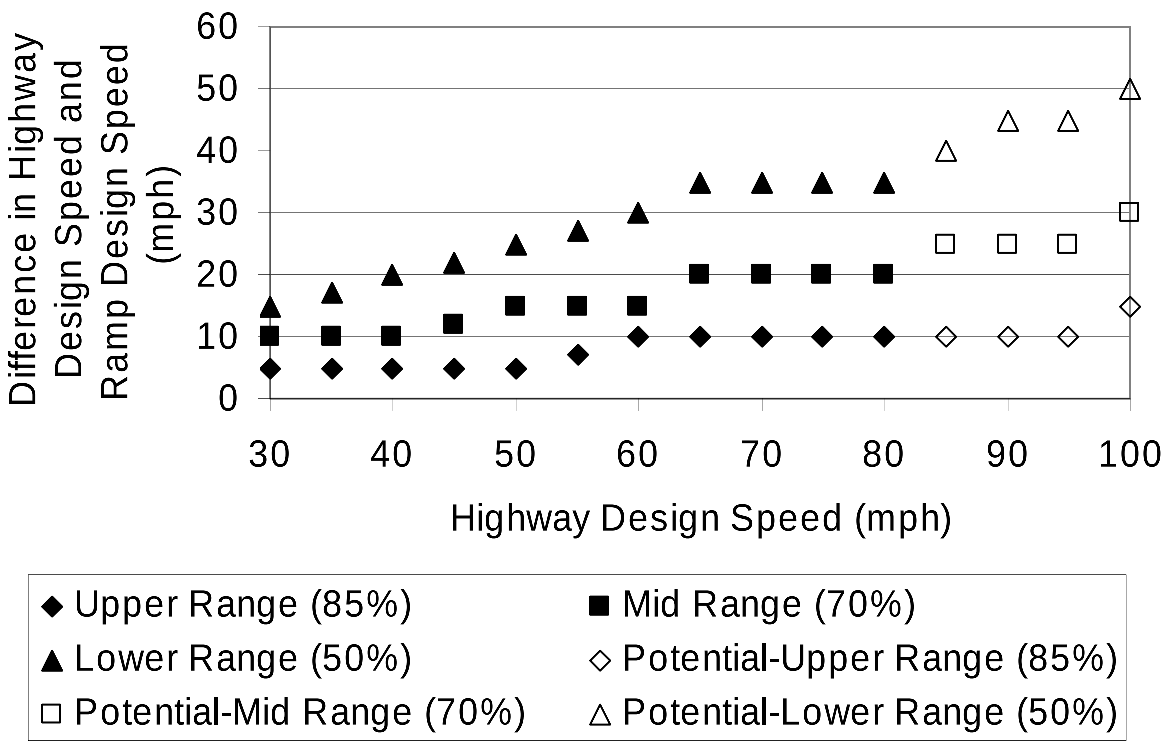
Figure 9-2. Plot of Calculated and Rounded Ramp Design Speeds Including Potential Ramp Design Speeds for Highway Design Speeds up to 100 mph [161 km/h].

A concern with the values in Table 9-3 and Figure 9-2 is the increasing absolute difference between the speeds on the highway and the speeds used for the ramp. Figure 9-3 plots the differences for the values shown in Table 9-3. For example, at the 100-mph [161 km/h] highway design speed the ramp design speed is 15 mph [24 km/h] less than the highway design speed for the upper range condition. A 10-mph [16 km/h] speed difference is the value used to consider the use of a truck climbing lane and was the value used when evaluating potential limits in grades. Therefore, another approach could be to set a cap on the maximum amount of speed difference between highway and ramp design speed at the three levels (upper, mid, lower). Suggested caps could be 10 mph for upper, 20 mph for mid, and 35 mph for lower [or 20 km/h for upper, 30 km/h for mid, and 60 km/h for lower]. These caps were determined using engineering judgment of the authors. The *Blue Book* includes suggested deceleration rates without brakes (24). These rates were extrapolated into the higher design speeds and used to determine the length of time to decelerate an amount equal to the suggested caps. In general, it takes under 3 seconds to drop 10 mph [16 km/h], between 4.7 and 5.8 seconds to drop 20 mph [32 km/h], and greater than 8 seconds to drop 35 mph [56 km/h] (see Table 9-4).

Potential ramp design speeds using the two approaches discussed above are shown in Table 9-5. For such high design speeds, the participants at the Roundtable Discussion Group suggested that the lower range (50 percent) criteria be removed from the design table.

Table 9-4. Number of Seconds to Reduce Speed without Brakes.

Design Speed (mph) [km/h]	Deceleration Rate without Brakes (mph/s, f/s^2 [m/s^2])	Seconds to Drop to Cap Speed of:		
		10 mph [16 km/h]	20 mph [32 km/h]	35 mph [56 km/h]
80 [129]	3.48, 5.12 [1.56]	2.88	5.76	10.07
85 [137]	3.66, 5.38 [1.64]	2.73	5.46	9.56
90 [145]	3.85, 5.66 [1.73]	2.60	5.20	9.09
95 [153]	4.04, 5.94 [1.81]	2.48	4.95	8.67
100 [161]	4.22, 6.20 [1.89]	2.37	4.74	8.29



Note: 1 mph = 1.61 km/h

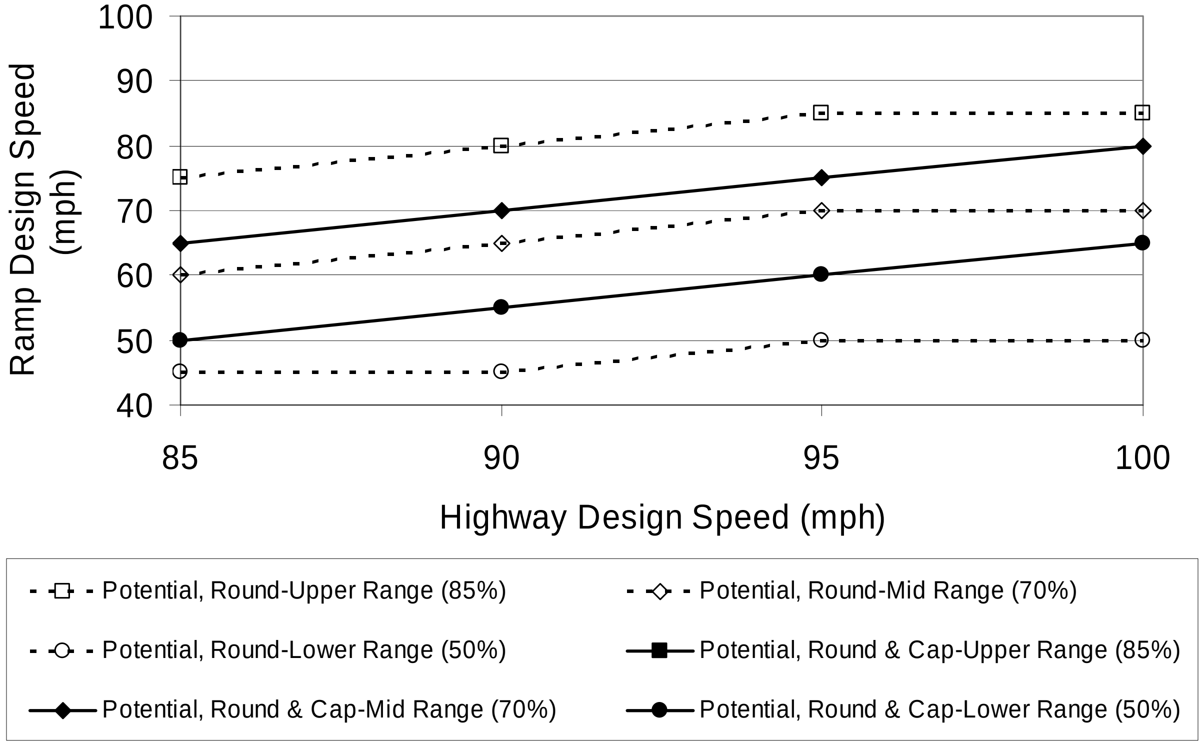
Figure 9-3. Plot of Absolute Differences between Highway Design Speed and Ramp Design Speeds.

Table 9-5. Potential Ramp Design Speeds.

(US Customary)								
Source-Range	**Values Determined Using Factors[a]**				**Values Determined Using Caps[b]**			
	Highway Design Speed (mph)							
	85	**90**	**95**	**100**	**85**	**90**	**95**	**100**
Calculated-Upper Range (85%)	72	77	81	85	72	77	81	85
Calculated-Mid Range (70%)	60	63	67	70	60	63	67	70
Calculated-Lower Range (50%)	43	45	48	50	43	45	48	50
Rounded-Upper Range (85%)	75	80	85	85	75	80	85	90
Rounded-Mid Range (70%)	60	65	70	70	65	70	75	80
Rounded-Lower Range (50%)[C]	45	45	50	50	50	55	60	75
Difference-Upper Range	3	4	4	0	3	4	4	5
Difference-Mid Range	1	2	4	0	5	8	7	10
Difference-Lower Range	3	0	3	0	7	10	12	25

(Metric)						
Source-Range	**Values Determined Using Factors[a]**			**Values Determined Using Caps[b]**		
	Highway Design Speed (km/h)					
	140	**150**	**160**	**140**	**150**	**160**
Calculated-Upper Range (85%)	119	128	136	119	128	136
Calculated-Mid Range (70%)	98	105	112	98	105	112
Calculated-Lower Range (50%)	70	75	80	70	75	80
Rounded-Upper Range (85%)	120	130	140	120	130	140
Rounded-Mid Range (70%)	100	110	120	110	120	130
Rounded-Lower Range (50%)[C]	70	80	80	80	90	100
Difference-Upper Range	1	3	4	1	3	4
Difference-Mid Range	2	5	8	12	15	18
Difference-Lower Range	0	5	0	10	15	20

[a] Values determined by calculating the 85, 70, or 50 percent value of the highway design speed and rounding up to nearest 5 mph increment.
[b] Values determined by calculating the 85, 70, or 50 percent value of the highway design speed and rounding up to nearest 5 mph [10 km/h] increment and then adjusting if the rounded value is more than the cap amount from the highway design speed (10 mph [20 km/h] for upper range, 20 mph [30 km/h] for mid range, and 35 mph [60 km/h] for lower range).
[c] Roundtable Discussion Group suggested that the lower range (50 percent) criteria be removed from the design table.
Shaded areas reflect high-design-speed potential values.



Note: 1 mph = 1.61 km/h

Figure 9-4. Potential Ramp Design Speeds.

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- Is it appropriate to use the lower range (50 percent) for designing ramps at high design speeds? What influences the decision on using upper range, mid range, and lower range criteria for ramp design and should those influences change with the use of higher highway design speed?
- Previous research that examined design speeds between 20 and 80 mph [32 and 129 km/h] has shown that an adequate margin of safety against both skidding and rollover exists as long as vehicles do not exceed the design speed of the curve *(49)*. However, under nearly worst-case conditions, skidding and rollover can occur on a horizontal curve, particularly at lower design speeds, if vehicles exceed the design speed by only a small amount. Deviations from assumed conditions that can increase the likelihood of skidding include the following:
    - vehicles traveling faster than the design speed,
    - vehicles turning more sharply than the curve radius,
    - lower pavement friction than assumed, and
    - poorer tires than assumed.

  Conditions that can include the likelihood of a rollover include:
    - vehicles traveling faster than the design speed,
    - vehicles turning more sharply than the curve radius, and

- trucks with a rollover threshold less than the assumed value of 0.35 g.

Additional research is needed to determine the potential for rollover and skidding for the higher design speeds, especially as applicable to ramp design.

CHAPTER 10

RAMP GRADES AND PROFILES

CURRENT GUIDANCE

The TRDM states the "tangent or controlling grade on ramps and direct connectors should be as flat as possible, and preferably should be limited to 4 percent or less (1)"

The *Green Book* states "ramp grades are not directly related to design speed; however, design speed is a general indication of the quality of design being used, and the gradient for a ramp with a high design speed should be flatter than for one with a low design speed. As general criteria, it is desirable that upgrades on ramps with a design speed of 45 to 50 mph [70 to 80 km/h] be limited to 3 to 5 percent." The *Green Book* also states "where appropriate for topographic conditions, grades steeper than desirable may be used. One-way downgrades on ramps should be held to the same general maximums, but in special cases they may be 2 percent greater." The *Green Book* notes "adequate sight distance is more important than a specific gradient control and should be favored in design" (*2*).

OTHER GUIDANCE

Minimum design criteria and guidelines were developed for TxDOT for use on the TTC-35 high-priority corridor (*11*). Their recommendations for grades are listed in Table 10-1.

Table 10-1. TTC-35 Design Criteria (*11*).

	Passenger Car Facility	**Truck Facility**
Direct Connectors	• 4 percent (usual max) • 6 percent (max uphill) • 6.5 percent (max downhill) • 0.50 percent (min)	• 3 percent (usual max) • 4 percent (max uphill) • 6.5 percent (max downhill) • 0.50 percent (min)
Ramps	• 3 percent (usual max) • 4 percent (max) • 0.50 percent (min)	• 4 percent (usual max) • 6 percent (max) • 0.50 percent (min)

The ITE *Freeway and Interchange Geometric Design Handbook* (50) includes guidelines for maximum grades for ramps (see Table 10-2).

Table 10-2. Guidelines, Maximum Grades for Design of Ramps from ITE *Freeway and Interchange Geometric Design Handbook* Table 10-10 (*50*).

Normal Conditions	**Heavy Truck Traffic**	**Rugged Terrain (Special Cases)**
4% to 6%	3% to 4%	6% to 8%

DISCUSSION

Truck abilities to maintain high speeds on grades were discussed in Chapter 3. For upgrades greater than 2 percent, a truck's speed is predicted to decrease by more than 10 mph [16 km/h] within 1850 ft [564 m].

In the TRDM, the value for the grade of a tangent or controlling grade on ramps and direct connector (4 percent or less) is provided independent of the mainlane design speed. Stated in another manner, the TRDM does not suggest different ramp grades for different mainlane design speeds (*1*).

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

The current guideline within the TRDM is to limit the controlling grade on ramps and direct connectors to 4 percent or less. The TRDM recommended maximum grade for an 80 mph [129 km/h] design speed is 3 percent on level terrain and 4 percent on rolling terrain (*1*). So the grade guideline for ramps (4 percent) is near the grade recommendation for 80 mph [129 km/h] design speed (3 or 4 percent).

Using a series of equations to predict truck speeds on grades resulted in the suggestion that 2 percent be considered as the maximum grade on the mainlanes for design speeds between 85 and 100 mph [137 and 161 km/h] (see Chapter 3). Based on those findings, limiting the controlling grade on ramps and direct connects to 2 percent or less, certainly no greater than 4 percent, would be in concert with existing criteria.

The suggestion of the participants of the Roundtable Discussion Group was to retain the current 4 percent guideline in the TRDM for the higher operating speeds.

Controls for minimum grades on ramps should be similar as for open highways.

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- While equations are available to predict speeds of vehicles on grades, the equations can result in speeds that appear to be less than the speeds observed in operations. Chapter 3 provides additional discussion on the evaluations using the available equations. Research is needed to verify truck and passenger car speeds for high speeds (85 to 100 mph [137 to 161 km/h]) on various grades. The research should also consider the merging and diverging behavior of vehicles near ramps while on upgrades or downgrades and the added concerns when the ramp includes a horizontal curve.

CHAPTER 11

RAMP CROSS SECTION AND CROSS SLOPE

CURRENT GUIDANCE

The TRDM states "superelevation rates, as related to curvature and design speed of the ramp or direct connector, are given in {Table 11-1}. While connecting roadways represent highly variable conditions, as high a superelevation rate as practicable should be used, preferably in the upper half or third of the indicated range, particularly in descending grades. Superelevation rates above 8 percent are shown in {Table 11-1} only to indicate the limits of the range. Superelevation rates above 8 percent are not recommended and a larger radius is preferable" (*1*). When the ramp design speed exceeds 45 mph [70 km/h], TRDM refers the designer to TRDM Tables 2-6 and 2-7, which are the primary superelevation tables for standard highway curves.

Per the TRDM, "the cross slope for portions of connecting roadways or ramps on tangent normally is sloped one way at a practical rate of 1.5 to 2 percent" (1). The maximum algebraic difference in pavement cross slope at connecting roadways or ramps should not exceed that set forth in Table 11-2. This guidance is echoed in the *Green Book* with discussion in the Grade Separations and Interchanges chapter, which refers the reader to the Intersection chapter.

OTHER GUIDANCE

Minimum design criteria and guidelines were developed for TxDOT for use on the TTC-35 high-priority corridor (*11*). Their recommendations for cross section elements are listed in Table 11-3.

The *Freeway and Interchange Geometric Design Handbook* (*50*) includes guidance on maximum algebraic difference in pavement cross slope that they stated were adapted from the *Green Book* (see Table 11-4).

Table 11-1. Superelevation Range for Curves on Connecting Roadways from TxDOT *Roadway Design Manual* (TRDM Table 3-21) (1).

(US Customary)						
Radius (ft)	**Range in Superelevation Rate (percent) for Connecting Roadways with Design Speed (mph) of:**					
	20	**25**	**30**	**35**	**40**	**45** [a]
90	2-10					
150	2-8	4-10				
230	2-6	3-8	6-10			
310	2-5	3-6	5-9	8-10		
430	2-4	3-5	4-7	6-9	9-10	
540	2-3	3-5	4-6	6-8	8-10	10-10
600	2-3	2-4	3-5	5-7	7-9	8-10
1000	2	2-3	3-4	4-5	5-6	7-9
1500	2	2	2-3	3-4	4-5	5-6
2000	2	2	2	2-3	3-4	4-5
3000	2	2	2	2	2-3	3-4
(Metric)						
Radius (m)	**Range in Superelevation Rate (percent) for Connecting Roadways with Design Speed (km/h) of:**					
	20	30	40	50	60	70 [a]
15	2-10					
25	2-9	2-10				
50	2-8	2-8	4-10			
80	2-6	2-6	3-8	6-10		
100	2-5	2-4	3-6	5-8		
115	2-3	2-4	3-6	5-8	8-10	
150	2-3	2-3	3-5	4-7	6-8	
160	2-3	2-3	3-5	4-7	6-8	10-10
200	2	2-3	2-4	3-5	5-7	6-8
300	2	2-3	2-3	3-4	4-5	5-7
500	2	2	2	2-3	3-4	4-5
700	2	2	2	2	2-3	3-4
1000	2	2	2	2	2	2-3

[a] See TRDM Tables 2-6 and 2-7 for design speeds greater than 45 mph [70 km/h].

Table 11-2. Maximum Algebraic Differences in Pavement Cross Slope at Connecting Roadway Terminals from TxDOT *Roadway Design Manual* (TRDM Table 3-22) and AASHTO *Green Book* (GB Exhibit 9-49) (1, 2).

(US Customary)		(Metric)	
Design Speed of Exit or Entrance Ramp (mph)	**Maximum Algebraic Difference in Cross Slope at Crossover Line (%)**	**Design Speed of Exit or Entrance Ramp (km/h)**	**Maximum Algebraic Difference in Cross Slope at Crossover Line (%)**
Less than or equal to 20	5.0 to 8.0	Less than or equal to 30	5.0 to 8.0
25 to 30	5.0 to 6.0	40 to 50	5.0 to 6.0
Greater than or equal to 35	4.0 to 5.0	Greater than or equal to 60	4.0 to 5.0

Table 11-3. Potential Criteria Developed for TTC-35.

	Passenger Car		Truck Facility	
	Direct Connectors	**Passenger Car Ramps**	**Direct Connectors**	**Passenger Car Ramps**
Horizontal Alignment – Minimum Radius	3405 ft (70 mph) [1039 m {113 km/h}] 2210 ft (60 mph) [674 m {97 km/h}]	4605 ft [1405 m]	4605 ft (80 mph) [1405 m {129 km/h}] 3405 ft (70 mph) [1039 m {113 km/h}]	3405 ft (70 mph) [1039 m {113 km/h}] 2210 ft (60 mph) [674 m {97 km/h}] 1055 (50 mph) [322 m {81 km/h}]
Horizontal Alignment – Maximum superelevation rate	0.06 ft/ft	0.06 ft/ft	0.06 ft/ft	0.06 ft/ft
Cross Slope (Across Entire Roadway) – 1 or 2 Ultimate Lanes	2.0% (min)	2.0% (min)	2.0% (min)	2.0% (min)
Cross Slope (Across Entire Roadway) – 3 Lanes	2.5% (min)	NA	2.5% (min)	2.5% (min)

Table 11-4. *Freeway and Interchange Geometric Design Handbook* Table 3-15: Maximum Algebraic Difference in Pavement Cross Slope for Design of Ramp Terminals at Crossover Crownline *(50)*.

Design Speed of Highway (mph [km/h])	Freeway/Highway Ramp Terminal	Ramp/Ramp Terminal
15-25 [24-40]	N/A	0.07
30-40 [48-64]	0.05	0.06
50-60 [81-97]	0.04	0.05
70-80 [113-129]	0.03	N/A

DISCUSSION

Cross Slope for Ramps

The cross slope for tangent sections of ramps should be similar to the cross slope for the mainlanes, and therefore should be in the 1.5 to 2.5 percent range, depending on pavement type and other local conditions. The cross slope for a ramp should be constant. The pavement should not have a crown in the middle of the ramp.

Superelevation Range for Ramp Curves

For most curves on ramps for high-design-speed roadways, especially ramps in a system interchange between two high-speed facilities, the curves on those ramps will be designed using TRDM Table 2-6 or 2-7 (i.e., as normal highway curves) rather than TRDM Table 3-21.

Therefore, TRDM Table 3-21 does not need to be updated for high-speed roadways, as it will likely not be used. For service interchanges (i.e., with surface streets at one end of the ramp), the curvature on the low-speed end of the ramp may be low enough to use TRDM Table 3-21, although TRDM Tables 2-6 and 2-7 contain the same information. Therefore, TRDM Tables 2-6 and 2-7 could be used for all ramp curves for high-design-speed roadways, even those curves with low design speeds.

Algebraic Difference in Cross Slope

The current TRDM guidance regarding the maximum algebraic difference in cross slope in Table 11-2 is taken directly from the *Green Book* Exhibit 9-49 (*2*). Exhibit 9-49 shows the maximum difference in cross slope between two adjacent lanes, one of which is a turn lane or turning roadway, for low-speed roadways. A minimum algebraic difference is not specified, although the minimum cannot be less than zero. Exhibit 9-49 is located in the *Green Book*'s Intersections chapter and was apparently developed specifically for the low-speed situations shown in the table. However, the *Green Book* refers directly to Exhibit 9-49 when it discusses the maximum algebraic cross slope difference for ramps and auxiliary lanes for roadways of any speed. Earlier versions of the AASHTO policies were reviewed to trace the source of the values. Similar values were included in the Intersection chapter of the *Blue Book* (24) published by the American Association of State Highway Officials (AASHO) in 1965. The values were in the *Blue Book* Table VII-14. No additional research could be found specifically on this topic to shed light on the origins of the table and limitations on its use.

Table 11-4 shows sensitivity to design speed in the algebraic difference in cross slope, with higher design speeds having a smaller algebraic difference. However, Table 11-4 was "adapted" from *Green Book* Exhibit 9-49. How the values in Table 11-4 were adopted is not stated, however, so it is unclear if there is any research supporting the selection of these values. Note, too, that 45-mph [72 km/h] and 65-mph [105 km/h] design speeds are not present in Table 11-4. They were omitted in the *Freeway and Interchange Geometric Design Handbook* document as well (50).

It is not clear whether a 4 to 5 percent difference in cross slope from the mainlanes to a ramp would be acceptable to drivers at high speeds. Even the 3 percent shown in Table 11-4 could be uncomfortable at speeds above 80 mph [129 km/h]. Desirably, the algebraic difference in cross slope would be as close to zero (i.e., a constant slope) as possible to prevent driver discomfort. If a change is necessary, then it should be restricted to the cross slope differences already present in a multilane freeway cross section (e.g., 0.5 to 1.0 percent). If the ramp terminal is located on a horizontal curve, it may not be reasonably possible to provide even a 1.0 percent cross slope break between the ramp and the mainlanes, and a larger cross slope break may be necessary. Further research is needed on this topic to assess how large an algebraic difference in cross slope that drivers would comfortably tolerate.

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

Cross Slope for Ramps

The cross slope for ramp tangent sections should be similar to the cross slope used on the mainlanes of the roadway. The cross slope on the ramp should be sloped in the same direction across the entire ramp.

Superelevation Range for Ramp Curves

The superelevation for ramp curves above 45 mph [72 km/h] should be determined from TRDM Tables 2-6 and 2-7 based on the curve's radius and the design speed of the ramp. For curves with design speeds below 45 mph [72 km/h], TRDM Tables 2-6 and 2-7 still may be used. TRDM Table 3-21 should not be used for ramp design for high-design-speed facilities.

Algebraic Difference in Cross Slope

At present, research on this topic was not found. Therefore, guidance on possible maximum algebraic cross slope differences for high-speed conditions is not available. Desirably, for driver comfort, the algebraic cross slope difference should be zero, and zero should be used wherever possible. Where zero is not practical, a difference of 1.0 percent appears to be a reasonable alternative, considering that the pavement may already have a cross slope difference of 0.5 to 1.0 percent for drainage purposes. The effect of a cross slope difference of 4 to 5 percent is not known, as was discussed in Chapter 7.

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- Further research on the effect of cross slope differences between lanes on vehicle behavior and driver comfort at high design speeds is needed. Specifically, the 4 percent difference across a roadway crown and the 4 to 5 percent difference allowed by the *Green Book* for ramps and auxiliary lanes need to be investigated.
- Verify the algebraic differences in grade values currently being used for intersections and turning roadways. Are these values representative of what is being used in design? Do they represent a comfortable and safe value for all users (for example, bicyclists and pedestrians in addition to vehicles)?

CHAPTER 12

DISTANCE BETWEEN SUCCESSIVE RAMPS

CURRENT GUIDANCE

The minimum acceptable distance between ramps is dependent upon the merge, diverge, and weaving operations that take place between ramps as well as distances required for signing. The TRDM states to see the *Highway Capacity Manual* (HCM) (*51*) for analysis of these requirements. A figure (reproduced as Figure 12-1) is provided to show the minimum distances between ramps for various ramp configurations. Key dimensions are:

Entrance Ramp Followed by Exit Ramp (see Figure 12-1 for control points)
- Minimum weaving length without auxiliary lane = 2000 ft [600 m]
- Minimum weaving length with auxiliary lane = 1500 ft [450 m]

Exit Ramp Followed by Exit Ramp
- Minimum distance 1000 ft [300 m]

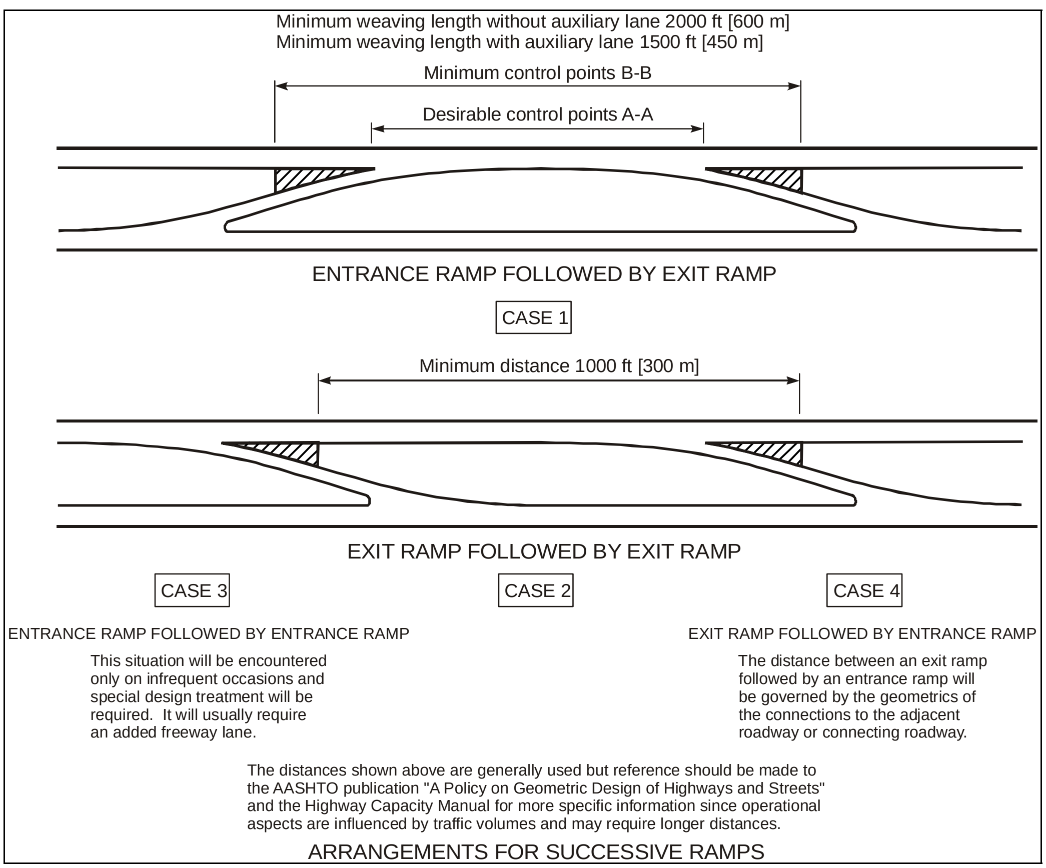


Figure 12-1. Arrangements for Successive Ramps from TxDOT *Roadway Design Manual* (TRDM Figure 3-37) (*1*).

OTHER GUIDANCE

Other key reference documents that provide information on ramp spacing, such as the *Green Book*, also encourage the reader to use the *Highway Capacity Manual* to identify appropriate spacing dimensions.

DISCUSSION

The *Green Book*'s guidance about ramp spacing is very similar to TRDM, although the *Green Book* does not specify a difference between ramp spacing with and without auxiliary lanes (*2*). The *Green Book* also recommends that users use the techniques for weaving sections provided in the *Highway Capacity Manual* to find the appropriate ramp spacing for design. Because of this recommendation, and the similarity of the *Green Book*'s advice to TRDM, the HCM weaving section analysis procedure will be discussed in some depth.

Highway Capacity Manual

The HCM weaving section analysis procedure is intended to show how well a particular weaving section will operate with given traffic volumes (51). The procedure itself is straightforward. First, the various parameters needed to perform the analysis are collected. These parameters include:

- segment length,
- total number of lanes in the weaving segment,
- number of lanes used by non-weaving vehicles,
- flow rates for each movement,
- the free-flow speed, and
- the number of lane changes required to complete the weaving maneuver.

From these parameters, a weaving diagram similar to the one shown in Figure 12-2 is developed, illustrating the weaving and non-weaving volumes measured in veh/hr. If the segment length is greater than 2500 ft, then the entrance and exit points are considered separately, not using the weaving analysis. Presumably, weaving is assumed to have no influence on traffic operation when the potential weaving section is longer than 2500 ft.

Next, the type of weaving section is determined. In the case of an entrance ramp followed by an exit ramp, the weaving section is defined to be Type A because entering and exiting vehicles must each make a lane change to complete the weaving maneuver. Types B and C weaving sections have one weaving movement that does not need to change lanes, while the other weaving movement must make either one lane change (Type B) or two or more lane changes (Type C).

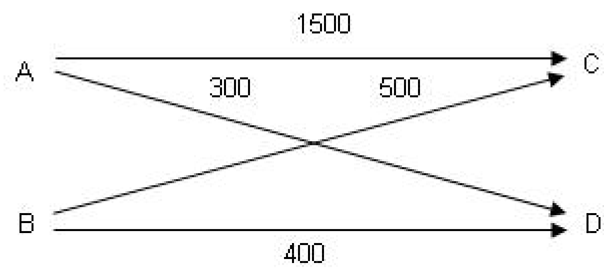


Figure 12-2. Example of a Weaving Diagram.

Then the average speed of weaving and non-weaving vehicles is calculated using the following equation:

$$S_i = 15 + \frac{S_{FF} - 10}{1 + W_i} \qquad \textbf{(12-1)}$$

Where:

S_i = average speed of weaving or non-weaving vehicles, mph;
S_{FF} = average free flow speed of the freeway segments entering and leaving the weaving segment, mph; and
W_i = weaving intensity factor.

The weaving intensity factor provides an adjustment to the average speed based on how much weaving occurs in the weaving segment. The larger the weaving intensity, the lower the average speeds. The weaving intensity is defined as:

$$W_i = \frac{a\left(1 + VR\right)^b \left(\frac{v}{N}\right)^c}{L^d} \qquad \textbf{(12-2)}$$

Where:

VR = volume ratio, ratio of weaving vehicles to total flow;
v = total flow rate in weaving segment;
N = total number of lanes in the weaving segment;
L = length of the weaving segment, ft; and
a, b, c, d = calibration constants.

The calibration constants a, b, c, and d vary depending on the type of operation and whether weaving or non-weaving speeds are being determined. Calibration constant a varies further depending on whether the weaving operation is "constrained" or "unconstrained." "Constrained" operations produce lower weaving speeds and higher non-weaving speeds than "unconstrained" operations due to limitations on the weaving space available in the "constrained" case. The calibration constants for the weaving intensity are provided in Exhibit 24-6 of the HCM. The constants for Type A operation only are repeated in Table 12-1.

Table 12-1. Constants for Computation of Weaving Intensity Factors for Type A Weaving Sections from Exhibit 24-6 of the *Highway Capacity Manual* (51).

Operation Type	Constants for Weaving Speed, S_w				Constants for Non-weaving Speed, S_{nw}			
	a	*b*	*c*	*d*	*a*	*b*	*c*	*d*
Unconstrained	0.15	2.2	0.97	0.80	0.0035	4.0	1.3	0.75
Constrained	0.35	2.2	0.97	0.80	0.0020	4.0	1.3	0.75

To determine the weaving intensity, the type of operation must be determined. Exhibit 24-7 in the HCM shows equations for each type of operation in terms of the number of lanes required for the weaving movements. For unconstrained Type A operation, the equation is:

$$N_w = 0.74(N)VR^{0.57}L^{0.234}/S_w^{0.438} \quad \textbf{(12-3)}$$

If N_w is greater than 1.4, then the weaving section has constrained operation. Otherwise, it is unconstrained.

Once the average speeds are determined, then the weaving segment speed is found using a weighted average of the weaving and non-weaving speeds. The weaving segment speed is then used to find the density of vehicles in the weaving segment, and the density is used to find the level of service of the segment.

The HCM procedure was designed as an operational tool for existing ramp configurations. As a result, it is difficult to back-solve a particular weaving section length from a level of service. The weaving and non-weaving volumes are also critical to the analysis but may not be known with any certainty prior to design, especially in the case of designing a new facility. These two considerations alone make it difficult to use the HCM weaving section procedure to determine ramp spacing for design. However, there are two more considerations that make the use of the HCM procedure essentially impossible. The following paragraphs outline these considerations.

The first consideration is the behavior of the "constrained" and "unconstrained" operation in Type A weaving sections. The HCM states that:

> "As the length of a Type A segment increases, constrained operation is more likely to result. As the length increases, the speed of weaving vehicles is also able to increase. Thus, weaving vehicles use more space as the length increases, and the likelihood of requiring more than the maximum of 1.4 lanes to achieve equilibrium also increases" (*51*).

This is a counter-intuitive idea. As more space is available for weaving, then the speeds of weaving and non-weaving vehicles would logically become closer, which would be more similar to "unconstrained" operation. Using the HCM procedure, short weaving sections (less than 1000 ft ramp spacing) may operate "unconstrained," depending on traffic conditions, while long weaving sections (2500 ft) may be "constrained" with the same traffic volumes. If "unconstrained" operation is the ideal state, then the HCM procedure indicates that ramps spaced at the minimum distances in the TRDM and *Green Book* are too far apart and should be closer.

Engineering judgment would indicate the opposite to be true. The other two types of weaving sections (B and C) behave in a manner more consistent with intuition.

The second consideration is the maximum length of weaving sections. A 2500-ft ramp spacing is always considered to be a Type A weaving section, possibly in "constrained" operation, depending on traffic conditions. However, a 2501-ft ramp spacing is considered to be two independent ramp junctions without weaving, which is effectively "unconstrained." Together with the pervious consideration, a very short ramp spacing (less than 1000 ft) would operate "unconstrained," even though weaving would be very difficult in that section and vehicle speeds may be severely restricted. A ramp spacing of between 1000 ft and 2500 ft could be "constrained" operation even though weaving and non-weaving speeds are becoming more similar, and the longer the ramp spacing, the more likely for the weaving section to have "constrained" operation. A ramp spacing of over 2500 ft would be effectively "unconstrained" again.

Some other apparent inconsistencies appeared while investigating the HCM weaving procedure. First, as the number of available lanes increases, the likelihood of "constrained" operation increases, even if the volumes do not change. Having more available through lanes should allow non-weaving traffic to avoid weaving traffic, increasing the likelihood of "unconstrained" operation instead. Second, as the non-weaving volume increases, the likelihood of "constrained" operation also decreases because the flow ratio decreases, even if there is no change in weaving section length. Higher non-weaving volumes should present more constraints on weaving rather than fewer. Third, higher free-flow speeds for non-weaving vehicles reduce the likelihood of "constrained" operation if no other parameters change. Higher non-weaving speeds should make it more difficult for weaving traffic to safely change speeds and merge.

These three apparent inconsistencies indicate that there may be considerable correlation among the calibration constants in the weaving intensity equation. Increased number of lanes is typically associated with increased volumes, especially non-weaving volumes. Increasing volumes and number of lanes are associated with urban conditions, which typically have lower free-flow speeds and also shorter ramp spacings than rural conditions. It is likely that the HCM procedure works under "usual" combinations of lanes, volumes, free-flow speeds, and ramp spacings, but quickly breaks down when one or more of these variables are unusual, such as when free-flow speeds exceed 80 mph or if more through lanes are available than expected.

If the transition between "unconstrained" and "constrained" operation were a smooth one, then the issue of "constrained" versus "unconstrained" operation would be moot. However, there is a considerable discontinuity between "constrained" and "unconstrained" operating speeds, and this discontinuity can occur with the change of a single vehicle or a single foot of segment length. For analyzing an existing weaving section, this situation is tolerable, because the entire range of conditions will not appear at one location. However, for analyzing possible configurations, the presence of the discontinuity means that the HCM weaving analysis procedure is not adequate.

Traffic Simulation

Because of the various issues with the HCM weaving analysis, it was determined that the HCM analysis could not be used to determine ramp spacings at high design speeds. Another alternative is the use of simulation. Traffic simulation of a freeway, with entrance and exit ramps spaced at various distances from each other, could show the behavior of traffic between the ramps and determine the most effective spacing for those ramps for various design speeds. Two simulation models were investigated for suitability: CORSIM and VISSIM. However, CORSIM could not accept free-flow speeds above 70 mph, so it was quickly eliminated from further consideration. VISSIM could handle the higher speeds. However, there were several additional unknowns that affect any simulation effort, including:

- realistic flow rates,
- truck percentages,
- calibration of speed distributions,
- range of ramp spacings to investigate, and
- lane configurations.

Some of these unknowns (e.g., range of ramp spacings) would have to be found by exploration using the simulation because there is no way to know them with any certainty before the study begins. The number of unknowns involved indicated to the project team that simulation would not be possible within the time, budget, and scope of this project. However, an investigation of a possible sensitivity of ramp spacing to design speed should be investigated with further research, especially for design speeds exceeding 80 mph.

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

Information is not readily available to support any changes to the material currently provided on ramp spacing. Additional investigation into appropriate spacing for design speeds of 85 to 100 mph [137 to 161 km/h] is needed.

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- Existing material does not indicate a relationship between spacing between ramps and design speed. However, it is logical to assume that a relationship exists. Research is needed to identify the spacing needed for different design (or operating) speeds and determine if one value is still appropriate.
- The location of signs for freeway exits is a function of roadway speed and will change with the increases in operating speeds on the high-design-speed roads. How the sign spacing interacts with ramp spacing also needs consideration.

CHAPTER 13

RAMP LANE AND SHOULDER WIDTHS

CURRENT GUIDANCE

The TxDOT *Roadway Design Manual* states in Chapter 3, Section 6, Freeways:

> "A 10 ft [3.0 m] outside shoulder should be maintained along all speed change lanes with a 6 ft [1.8 m] shoulder considered in those instances where light weaving movements take place. See Table {13-1} for future information" (*1*).

Table 13-1. Roadway Widths for Controlled Access Facilities from TxDOT *Roadway Design Manual* (TRDM Table 3-18) (*1*).

(US Customary)			
Type of Roadway	**Inside Shoulder Width (ft)**	**Outside Shoulder Width[b] (ft)**	**Traffic Lanes (ft)**
Mainlanes–4-lane divided	4	10	24
Mainlanes–6-lane or more divided	10	10	36[a]
1-lane direct connect[b]	2 (roadway) 4 (street)	8	14
2-lane direct connect	2 (roadway) 4 (street)	8	24
Ramps[b] (uncurbed)	2 (roadway) 4 (street)	6 (min), 8 (des)	14
Ramps[c] (curbed)	--	--	22
(Metric)			
Type of Roadway	**Inside Shoulder Width (m)**	**Outside Shoulder Width[b] (m)**	**Traffic Lanes (m)**
Mainlanes–4-lane divided	1.2	3.0	7.2
Mainlanes–6-lane or more divided	3.0	3.0	10.8[a]
1-lane direct connect[b]	0.6 (roadway) 1.2 (street)	2.4	4.2
2-lane direct connect	0.6 (roadway) 1.2 (street)	2.4	7.2
Ramps[b] (uncurbed)	0.6 (roadway) 1.2 (street)	1.8 (min), 2.4 (des)	4.2
Ramps[c] (curbed)	--	--	6.6

[a] For more than six lanes, add 12 ft [3.6 m] width per lane.
[b] If sight distance restrictions are present due to horizontal curvature, the shoulder width on the inside of the curve may be increased to 8 ft [2.4 m] and the shoulder width on the outside of the curve decreased to 2 ft [0.6 m] (roadway) or 4 ft [1.2 m] (street).
[c] The curb for a ramp lane will be mountable and limited to 4 inches [100 mm] or less in height. The width of the curbed ramp lane is measured face to face of curb. Existing curb ramp lane widths of 19 ft [5.7 m] may be retained.

The *Green Book* includes more discussion on ramp traveled-way widths and begins by stating that the widths "are governed by the type of operation, curvature, and volume and type of traffic." Widths are provided for three general design traffic conditions summarized as a) predominately passenger cars, b) sufficient single-unit vehicles to govern design, and c) sufficient buses and combination trucks to govern design (*2*).

OTHER GUIDANCE

Minimum design criteria and guidelines were developed for TxDOT for use on the TTC-35 high-priority corridor (*11*). Table 13-2 lists the criteria.

Table 13-2. Potential Criteria Developed for TTC-35 (*11*).

(US Customary)				
	Passenger Car		Truck Facility	
	Direct Connectors	Ramps	Direct Connectors	Ramps
Lane Width	14 ft (12 ft for two lanes)	14 ft (12 ft for two lanes)	14 ft (13 ft for two lanes)	14 ft (13 ft for two lanes)
Shoulder Width, Inside Shoulder	8 ft (one lane) 4 ft (two lanes)[a]	4 ft[b]	8 ft (one lane) 4 ft (two lane)[a]	4 ft[b]
Shoulder Width, Outside Shoulder	8 ft[a]	6 ft[b]	8 ft[a]	6 ft[b]
(Metric)				
	Passenger Car		**Truck Facility**	
	Direct Connectors	**Ramps**	**Direct Connectors**	**Ramps**
Lane Width	4.3 m (3.7 m for two lanes)	4.3 m (3.7 m for two lanes)	4.3 m (4.0 m ft for two lanes)	4.3 m (4.0 m for two lanes)
Shoulder Width, Inside Shoulder	2.4 m (one lane) 1.2 m (two lanes)[a]	1.2 m[b]	2.4 m (one lane) 1.2 m (two lane)[a]	1.2 m[b]
Shoulder Width, Outside Shoulder	2.4 m[a]	1.8 m[b]	2.4 m[a]	1.8 m[b]

[a] To mitigate restrictions on the design imposed by sight distance, it is acceptable to position the 8-ft [2.4 m] shoulder on the inside of the curve and the 4-ft [1.2 m] shoulder on the outside of the curve for two-lane direct connectors.
[b] Shoulder width measured from lane line to face of curb. Total ramp width from face of curb to face of curb is 24 ft [7.3 m] (one lane) and 34 ft [10.4 m] (two lanes).

DISCUSSION

A recent TxDOT project (*7*) evaluated then-current (early 2000s) geometric design criteria to determine whether the criteria adequately reflected truck characteristics. A recommendation that relates to lane/shoulder width on a ramp includes increasing the offset between the outer edge of the usable shoulder and vertical elements such as barriers by a minimum of 2 ft [0.6 m].

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

Table 13-3 lists the suggested ramp width values for high design speeds. The truck facility dimensions for shoulders were determined from adding 2 ft [3.2 m] to the values identified for passenger cars based on the advice provided in a previous TxDOT project (*7*). The increased ramp lane widths for a truck facility reflect the wider lane widths suggested for mainlanes (see Chapter 5).

Those in attendance at the Roundtable Discussion Group meeting expressed a preference to only have one set of criteria rather than having a separate set for high truck volume facilities. In addition, the selected widths are to consider the potential for vehicle breakdowns and the need to provide passing opportunities. The refined suggested ramp values are provided in Table 13-4.

Table 13-3. Preliminary Ramp Width Values for High Design Speeds.

(US Customary)			
Type of Roadway	**Inside Shoulder Width (ft)**	**Outside Shoulder Width[a] (ft)**	**Traffic Lanes (ft)**
Passenger Car or Mixed-Use Facility			
1-lane direct connect	8	8	14
2-lane direct connect[a]	4	8	24
Ramps[a] (uncurbed)	8 (one lane), 4 (two lanes)	6 (min), 8 (des)	14
Truck or High Truck Volumes[b] Facility			
1-lane direct connect	10	10	15
2-lane direct connect[c]	6	10	26
Ramps[c] (uncurbed)	10 (one lane), 6 (two lanes)	8 (min), 10 (des)	15
(Metric)			
Type of Roadway	**Inside Shoulder Width (m)**	**Outside Shoulder Width[a] (m)**	**Traffic Lanes (m)**
Passenger Car or Mixed-Use Facility			
1-lane direct connect	2.4	2.4	4.3
2-lane direct connect[a]	1.2	2.4	7.3
Ramps[a] (uncurbed)	2.4 (one lane), 1.2 (two lanes)	1.8 (min), 2.4 (des)	4.3
Truck or High Truck Volumes[b] Facility			
1-lane direct connect	3.1	3.1	4.6
2-lane direct connect[c]	1.8	3.1	7.9
Ramps[c] (uncurbed)	3.1 (one lane), 1.8 (two lanes)	2.4 (min), 3.1 (des)	4.6

[a] If sight distance restrictions are present due to horizontal curvature, the shoulder width on the inside of the curve may be increased to 8 ft [2.4 m] and the shoulder width on the outside of the curve decreased to 4 ft [1.2 m] (two lane).
[b] High truck volumes occur when the directional design hourly volume for truck traffic exceeds 250 veh/h.
[c] If sight distance restrictions are present due to horizontal curvature, the shoulder width on the inside of the curve may be increased to 10 ft [3.1 m] and the shoulder width on the outside of the curve decreased to 6 ft [1.2 m] (two lane).

Table 13-4. Potential Ramp Width Values for High Design Speeds.

(US Customary)			
Number of Lanes on Ramp	**Inside Shoulder Width (ft)**	**Outside Shoulder Width (ft)**	**Traffic Lanes (ft)**
1 lane	8	10	14
2 lanes	4	10	26
(Metric)			
Number of Lanes on Ramp	**Inside Shoulder Width (m)**	**Outside Shoulder Width (m)**	**Traffic Lanes (m)**
1 lane	2.4	3.0	4.3
2 lanes	1.2	3.0	7.9

Note: If sight distance restrictions are present due to horizontal curvature, the shoulder width on the inside of the curve may be increased to 10 ft [3.0 m] and the shoulder width on the outside of the curve decreased to 8 ft [2.4 m] for 1-lane or 4 ft [1.2 m] for 2-lane ramp.
Shaded areas reflect high-design-speed potential values.

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- The amount of lane widening needed on horizontal curves could impact the width of ramp. Additional investigation is needed on this topic.
- The TRDM does not include different ramp width values for different traffic conditions, which the *Green Book* does. With the consideration of designing and building truck-exclusive facilities, should ramp widths be a function of the expected percentage or amount of trucks? If so, at what frequency of trucks should the wider dimensions be used? (This research need is common to several criteria, for example, lane and shoulder width on the mainlanes.)
- The TRDM Table 3-18 includes guidance for ramps with curbs (currently can have a mountable curb limited to 4 inches [102 mm] or less in height). Should a curb be used on a high-design-speed ramp?

CHAPTER 14

ACCELERATION AND DECELERATION RAMP LENGTHS

CURRENT GUIDANCE

The TRDM provides design criteria for exit and entrance ramp acceleration, deceleration, and taper lengths (*1*). Table 14-1 provides the lengths and Table 14-2 provides the adjustment factors for deceleration. Table 14-3 provides the lengths and Table 14-4 provides the adjustment factors for acceleration. Similar values are provided in the 2004 *Green Book.*

Table 14-1. Lengths of Exit Ramp Speed Change Lanes from TxDOT *Roadway Design Manual* (TRDM Figure 3-36) (*1*).

(US Customary)										
Highway Design Speed (mph)	Minimum Length of Taper, T (ft)	Deceleration Length, D (ft) for Exit Curve Design Speed (mph)								
		Stop	15	20	25	30	35	40	45	50
		And Initial Speed (mph)								
		0	14	18	22	26	30	36	40	44
30	150	235	200	170	140	--	--	--	--	--
35	165	280	250	210	185	150	--	--	--	--
40	180	320	295	265	235	185	155	--	--	--
45	200	385	350	325	295	250	220	--	--	--
50	230	435	405	385	355	315	285	225	175	--
55	250	480	455	440	410	380	350	285	235	--
60	265	530	500	480	460	430	405	350	300	240
65	285	570	540	520	500	470	440	390	340	280
70	300	615	590	570	550	520	490	440	390	340
75	330	660	635	620	600	575	535	490	440	390
Note: Where providing desirable deceleration length is impractical, it is acceptable to allow for a moderate amount of deceleration (10 mph) within the through lanes and to consider the taper as part of the deceleration length.										
(Metric)										
Highway Design Speed (km/h)	Minimum Length of Taper, T (m)	Deceleration Length, D (m) for Exit Curve Design Speed (km/h)								
		Stop	20	30	40	50	60	70	80	--
		And Initial Speed (km/h)								
		0	20	28	35	42	51	63	70	--
50	45	75	70	60	45	--	--	--	--	--
60	55	95	90	80	65	55	--	--	--	--
70	60	110	105	95	85	70	55	--	--	--
80	70	130	125	115	100	90	80	55	--	--
90	75	145	140	135	120	110	100	75	60	--
100	80	170	165	155	145	135	120	100	85	--
110	90	180	180	170	160	150	140	120	105	--
120	100	200	195	185	175	170	155	140	120	--
Note: Where providing desirable deceleration length is impractical, it is acceptable to allow for a moderate amount of deceleration (15 km/h) within the through lanes and to consider the taper as part of the deceleration length.										

Table 14-2. Speed Change Lane Adjustment Factors as a Function of a Grade for Deceleration Lanes from TxDOT *Roadway Design Manual* (TRDM Figure 3-14) (*1*).

Design Speed of Roadway (mph or km/h)	Ratio of Length on Grade to Length on Level[a]			
	3 to 4% Upgrade	3 to 4% Downgrade	5 to 6% Upgrade	5 to 6% Downgrade
All	0.9	1.2	0.8	1.35
[a] Ratio in this table multiplied by length of deceleration distances gives length of deceleration distance on grade.				

Table 14-3. Lengths of Entrance Ramp Speed Change Lanes from TxDOT *Roadway Design Manual* (TRDM Figure 3-36) (*1*).

(US Customary)										
Highway Design Speed (mph)	Minimum Length of Taper, T (ft)	Acceleration Length, A (ft) for Entrance Curve Design Speed (mph)								
		Stop	15	20	25	30	35	40	45	50
		And Initial Speed (mph)								
		0	14	18	22	26	30	36	40	44
30	150	180	140	--	--	--	--	--	--	--
35	165	280	220	160	--	--	--	--	--	--
40	180	360	300	270	210	120	--	--	--	--
45	200	560	490	440	380	280	160	--	--	--
50	230	720	660	610	550	450	350	130	--	--
55	250	960	900	810	780	670	550	320	150	--
60	265	1200	1140	1100	1020	910	800	550	420	180
65	285	1410	1350	1310	1220	1120	1000	770	600	370
70	300	1620	1560	1520	1420	1350	1230	1000	820	580
75	330	1790	1730	1630	1580	1510	1420	1160	1040	780
Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1300 ft.										
(Metric)										
Highway Design Speed (km/h)	Minimum Length of Taper, T (m)	Acceleration Length, A (m) for Entrance Curve Design Speed (km/h)								
		Stop	20	30	40	50	60	70	80	--
		And Initial Speed (km/h)								
		0	20	28	35	42	51	63	70	--
50	45	60	50	30	--	--	--	--	--	--
60	55	95	80	65	45	--	--	--	--	--
70	60	150	130	110	90	65	--	--	--	--
80	70	200	180	165	145	115	65	--	--	--
90	75	260	245	225	205	175	125	35	--	--
100	80	345	325	305	285	255	205	110	40	--
110	90	430	410	390	370	340	290	200	125	--
120	100	545	530	515	490	460	410	325	245	--
Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 400 m.										

Table 14-4. Speed Change Lane Adjustment Factors as a Function of a Grade for Acceleration Lanes from TxDOT *Roadway Design Manual* (TRDM Figure 3-14) (1).

(US Customary)

Design Speed of Roadway (mph)	Ratio of Length on Grade to Length on Level[a]							
	20	25	30	35	40	45	50	All Speeds
	3 to 4% Upgrade							3 to 4% Downgrade
35	--	--	--	--	--	--	--	0.7
40	1.3	1.3	1.3	1.3	--	--	--	0.7
45	1.3	1.3	1.35	1.35	--	--	--	0.675
50	1.3	1.35	1.4	1.4	1.4	--	--	0.65
55	1.35	1.4	1.45	1.45	1.45	1.45	--	0.625
60	1.4	1.45	1.5	1.5	1.5	1.55	1.6	0.6
65	1.45	1.5	1.55	1.55	1.6	1.65	1.7	0.6
70	1.5	1.55	1.6	1.65	1.7	1.75	1.8	0.6
75	1.55	1.6	1.65	1.7	1.75	1.8	1.9	0.6
80	1.6	1.65	1.7	1.75	1.8	1.9	2.0	0.6
	5 to 6 % Upgrade							5 to 6 % Downgrade
35	--	--	--	--	--	--	--	0.6
40	1.5	1.5	1.5	1.6	--	--	--	0.6
45	1.5	1.55	1.6	1.6	--	--	--	0.575
50	1.5	1.6	1.7	1.8	1.9	2.0	--	0.55
55	1.6	1.7	1.8	1.9	2.05	2.1	--	0.525
60	1.7	1.8	1.9	2.05	2.2	2.4	2.5	0.5
65	1.85	1.95	2.05	2.2	2.4	2.6	2.75	0.5
70	2.0	2.1	2.2	2.4	2.6	2.8	3.0	0.5
75	2.2	2.3	2.4	2.65	2.9	3.2	3.5	0.5
80	2.4	2.5	2.9	2.9	2.9	3.6	4.0	0.5

(Metric)

Design Speed of Roadway (km/h)	Ratio of Length on Grade to Length on Level[a]					
	40	50	60	70	80	All Speeds
	3 to 4% Upgrade					3 to 4% Downgrade
60	1.3	1.4	1.4	--	--	0.7
70	1.3	1.4	1.4	1.5	--	0.65
80	1.4	1.5	1.5	1.5	1.6	0.65
90	1.4	1.5	1.5	1.5	1.6	0.6
100	1.5	1.6	1.7	1.7	1.8	0.6
110	1.5	1.6	1.7	1.7	1.8	0.6
120	1.5	1.6	1.7	1.7	1.8	0.6
130	1.5	1.6	1.7	1.7	1.8	0.6
	5 to 6 % Upgrade					5 to 6 % Downgrade
60	1.5	1.5	--	--	--	0.6
70	1.5	1.6	1.7	--	--	0.6
80	1.5	1.7	1.9	1.8	--	0.55
90	1.6	1.8	2.0	2.1	2.2	0.55
100	1.7	1.9	2.2	2.4	2.5	0.5
110	2.0	2.2	2.6	2.8	3.0	0.5
120	2.3	2.5	3.0	3.2	3.5	0.5
130	2.6	2.8	3.4	3.6	4.0	0.5

[a] Ratio in this table multiplied by length of acceleration distances gives length of acceleration distance on grade.

OTHER GUIDANCE

The ITE *Freeway and Interchange Geometric Design Handbook* (50) includes deceleration lengths (see Table 14-5). The values are "transition guidelines longitudinal dimensions from the painted exit gore to the controlling ramp curve." The document also includes reproductions of the *Green Book* deceleration and acceleration figures in another chapter. The reason for the differences between the values in their Figure 6-18 (see Table 14-5) and the 2004 *Green Book* values are not provided.

Table 14-5. Ramp Transition Lengths from ITE *Freeway and Interchange Geometric Design Handbook* (ITE Figure 6-18) (50).

(US Customary)				
Freeway Design Speed (mph)	**Ramp Transition Lengths (ft) for Ramp/Controlling Curve Design Speed (mph) ITE Value {TRDM/2004 *Green Book* Values, for comparison}**			
	50	**40**	**30**	**25**
60	200 {240}	275 {350}	350 {430}	450 {460}
70	225 {340}	325 {440}	425 {520}	550 {550}
(Metric)				
Freeway Design Speed (km/h)	**Ramp Transition Lengths (m) for Ramp/Controlling Curve Design Speed (km/h) ITE Value {TRDM/2004 *Green Book* Values, for comparison}**			
	81	**64**	**48**	**40**
97	61 {73}	84 {107}	107 {131}	137 {140}
113	69 {104}	99 {134}	130 {159}	168 {168}

NCHRP Report 505 (*13*) investigates the weight-to-power ratios implied by the acceleration lengths listed in Table 14-1. The analysis indicated that the minimum lengths of acceleration lanes presented in the *Green Book* may be sufficient to accommodate average trucks but not to accommodate heavily loaded trucks. For example for a 0 percent grade, trucks with weight-to-power ratios in the range of 100 to 145 lb/hp [61 to 88 kg/kw] have sufficient acceleration capabilities to achieve the given speeds within the minimum acceleration length. For 2 percent grades, the weight-to-power ratios are in the range of 65 to 110 lb/hp [40 to 67 kg/kw]. The analysis indicated that the underlying assumptions for estimating the minimum acceleration lengths do not necessarily account for the performance capabilities of heavily loaded vehicles.

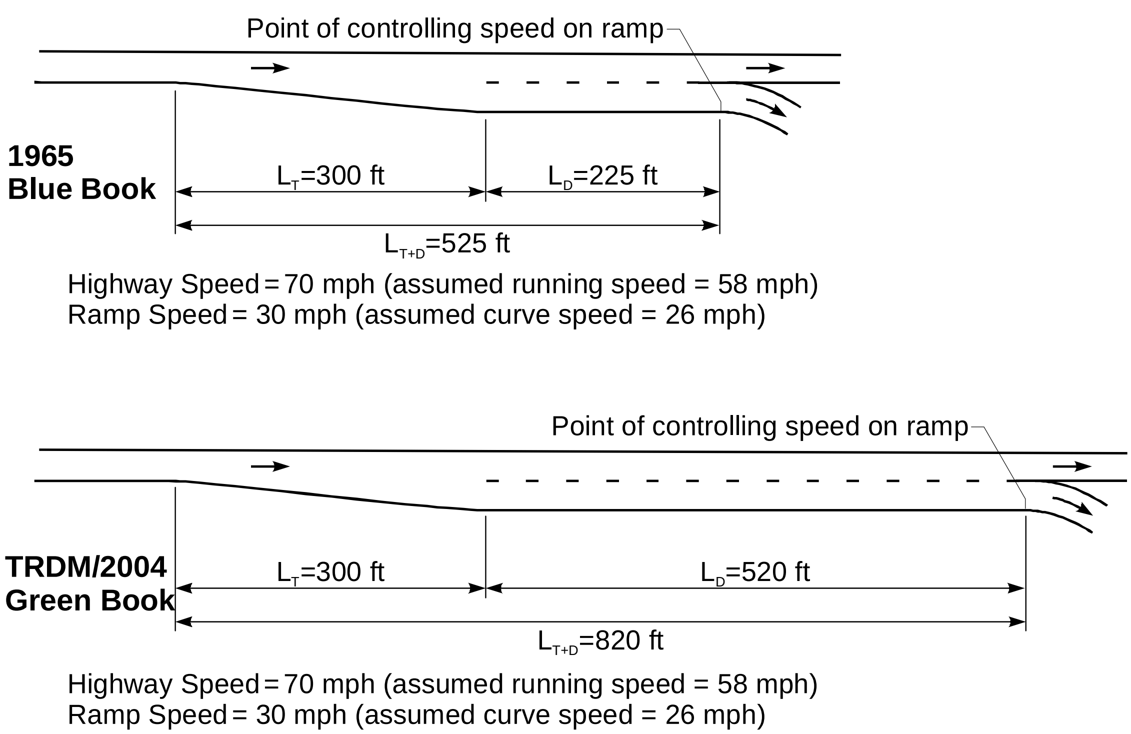
Although the sensitivity analyses presented in NCHRP Report 505 indicated a potential need to increase acceleration lengths to accommodate heavily loaded trucks better, crash data did not show that trucks have difficulties with acceleration lanes designed according to the AASHTO criteria. Therefore, no change was recommended because there was no indication that trucks are encountering specific problems on acceleration lanes designed in accordance with the *Green Book* criteria. Since the findings from NCHRP Report 505, NCHRP sponsored project 15-31 (52) to develop improved design guidance for freeway mainline ramp terminals suitable for inclusion in the AASHTO *Green Book*.

DISCUSSION

Deceleration Lengths

Deceleration lengths similar to the values in the TRDM (see Table 14-1) were included in the 1965 *Blue Book* (*24*). Deceleration length values are also included in the *Policies on Geometric Highway Design* published by AASHO in 1954 (*4*). These values are not as close to the TRDM values as the 1965 *Blue Book* values. Therefore, comparisons will be made to the 1965 *Blue Book* values. To extend the values into higher speeds, the equations and assumptions used to generate the values currently in the TRDM are needed. Assuming that the source of the TRDM values is the similar *Blue Book* values, the research team attempted to reproduce the *Blue Book* values. While the values appear to be similar, a key difference is that the deceleration lengths in the *Blue Book* **included** the length of the taper while the TRDM lists the minimum length of taper separately. Figure 14-1 illustrates the differences.

The AASHTO Policy on Design of Urban Highways and Arterial Streets (*Red Book*) (*53*) published in 1973 includes minimum deceleration lengths for exit terminals (1973 *Red Book* Table J-10). Accompanying the lengths is a graphic that shows the deceleration dimension as beginning at the end of the taper rather than including the taper. Therefore, the separation of the taper length dimension from the deceleration dimension occurred between 1965 and 1973. However, the reason for the separation is not apparent.



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 14-1. Illustration of Difference in Measurements on Exit Ramp for TRDM/2004 *Green Book* and 1965 *Blue Book* (*1, 2, 24*).

The 1965 *Blue Book* provided the deceleration length values derived (1965 *Blue Book* Figure VII-16) along with rounded values for use in design (1965 *Blue Book* Table VII-10) *(24)*. The deceleration dimensions in the 1973 *Red Book* match the values shown in the table of the 1965 *Blue Book* that provides for the calculated lengths (1965 *Blue Book* Figure VII-16) rather than the rounded lengths (1965 *Blue Book* Table VII-10). The TRDM values match the values in the 2004 *Green Book*. For some combinations, the TRDM (and the 2004 *Green Book*) deceleration lengths differ slightly from the 1965 *Blue Book* values. The TRDM/2004 *Green Book* also include values for 35-, 45-, and 55-mph [56, 72, and 89 km/h] design speeds but not the 80-mph [129 km/h] design speed that was included in the 1965 *Blue Book*. The differences between the TRDM/2004 *Green Book* and the 1965 *Blue Book* values are listed in Table 14-6. The largest difference was for a 75-mph [121 km/h] highway design speed to a 40-mph [64 km/h] exit curve design speed. The TRDM and the 2004 *Green Book* have a 490-ft [149 m] deceleration length while the 1965 *Blue Book* has a 470-ft [143 m] dimension, a 20-ft [6 m] difference.

Table 14-6. Lengths of Exit Ramp Speed Change Lanes for TRDM/2004 *Green Book* and 1965 *Blue Book* (*1, 2, 24*).

Highway Design Speed (mph)	Difference in Deceleration Length (ft) between TRDM/2004 *Green* Book and 1965 *Blue* Book for Exit Curve Design Speed (mph)								
	Stop	15	20	25	30	35	40	45	50
	Assumed Exit Curve Speed (mph)								
	0	14	18	22	26	30	36	40	44
30	0	15	10	0	--	--	--	--	--
35	NB	NB	NB	NB	NB	--	--	--	--
40	5	0	0	0	0	0	--	--	--
45	NB	NB	NB	NB	NB	NB	--	--	--
50	0	0	0	0	0	0	0	0	--
55	NB	NB	NB	NB	NB	NB	NB	NB	--
60	0	0	-10	0	0	-5	10	0	0
65	0	0	-10	10	-10	10	10	10	0
70	0	0	0	0	10	0	10	0	0
75	0	5	10	10	15	5	20	0	0
80	NT	NT	NT	NT	NT	NT	NT	NT	NT
NB = value not included in 1965 *Blue Book*. NT = value not included in TRDM.									
Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m									

The 1965 *Blue Book* states that the length of the deceleration lane is based on three factors in combination:

A. the speed at which drivers maneuver onto the auxiliary lane,
B. the speed at which drivers turn after traversing the deceleration lane, and
C. the manner of decelerating or the deceleration factors.

For factor A, the 1965 *Blue Book* states that "most drivers travel at a speed not greater than the average running speed of the highway" *(24)*. So on a freeway with a 70-mph [113 km/h] design speed, the assumption is that a driver will enter the auxiliary lane at 58 mph [93 km/h] (see Table 14-7) – a 12-mph [19 km/h] reduction. Previous research *(54)* has demonstrated that speeds on rural two-lane highways routinely have 85th percentile speeds in excess of the inferred design speed of tight horizontal curves. Other research (55) has demonstrated that 85th percentile operating speed exceeds posted speed on arterial streets and rural highways. Therefore, it would be logical to also assume that drivers may exceed the design speed or the posted speed on freeways. While data on speeds during maneuver onto an auxiliary lane are not readily available, the assumption that drivers will decelerate on a freeway prior to entering the ramp needs to be investigated.

Table 14-7. Running Speeds for Horizontal Curves.

(US Customary)				(Metric)	
Design Speed (mph)	**Average Running Speed (mph) from 1965 *Blue Book*[a]**		**Running Speed from 2004 *Green Book* and Extrapolated Values[b] (mph)**	**Design Speed (km/h)**	**Running Speed from 2004 *Green Book* and Extrapolated Values[b] (km/h)**
	Intersections Curve (1965 *Blue Book* Table VII-3) and used for Exit Curve	Highway Average Running Speed (1965 *Blue Book* Figure VII-16)			
15	14		15	20	20
20	18		20	30	30
25	22		24	40	40
30	26	28	28	50	47
35	30	32 [c]	32	60	55
40	36	36	36	70	63
45	40	40 [c]	40	80	70
50	44	44	44	90	77
55		48 [c]	48	100	85
60		52	52	110	91
65		55	55	120	98
70		58	58	130	102
75		61	61	140	110
80		64	64	150	118 [d]
85			67	160	131 [d]
90			70	--	--
95			75 [d]	--	--
100			82 [d]	--	--

[a] For design speeds of 15 to 40 mph, values are from 1965 *Blue Book* Table VII-3, for design speeds greater than 40 mph, values are from 1965 *Blue Book* Figure VII-16.
[b] Extrapolated values are from Chapter 8.
[c] Value estimated as average of two neighboring design speeds.
[d] Values adjusted to eliminate negative friction on curve (see Horizontal Alignment and Superelevation section in Chapter 8).
Shaded areas reflect high-design-speed potential values.

Similar concerns exist for factor B – the speed at the controlling curve after traversing the deceleration lane. The procedure assumes that drivers will begin the curve at a speed less than the curve's design speed. This conservative assumption is not currently supported with research findings or operational experience.

While the assumptions made within factors A and B may be questionable, their values are clearly provided in the 1965 *Blue Book.* For factor C – the manner of decelerating or the deceleration factors – the values are not as clear. The 1965 *Blue Book* states that deceleration is a two-step process: first, the accelerator pedal is released (assumed for 3 seconds) and the vehicle slows in gear without the use of brakes, and second, the brakes are applied. Two graphs are included in the 1965 *Blue Book* (Figure VII-15) to provide these distances. It appears that the graphs were based on data from studies conducted in the 1930s and 1940s. The *Blue Book* states:

> "a comfortable overall rate of deceleration while braking from 70 to a complete stop has been found to be about 6.2 mph per second (9 feet per second, per second)…In applying this rate at approaches to intersections, it is logical to assume that it decreases as the approach speed is lowered in a manner similar to that found in approaching a stop sign. Accordingly, the overall deceleration rate is assumed to vary from 6.2 mph per second (f = 0.28) for initial speed of 70 mph to 4 mph per second (f = 0.18) for initial speed of 30 mph" (*24*).

Using the above information, the equations to calculate deceleration lengths are:

<u>US Customary</u>

$$D = 1.47V_h t_n - 0.5d_n\left(t_n\right)^2 + \frac{\left(1.47V_r\right)^2 - \left(1.47V_a\right)^2}{2d_{wb}} \qquad \textbf{(14-1, US)}$$

$$V_a = \frac{1.47V_h + d_n t_n}{1.47} \qquad \textbf{(14-2, US)}$$

Where:

- V_h = Highway speed, mph;
- V_a = Speed after t_n seconds of deceleration without brakes, mph;
- V_r = Entering speed for controlling exit ramp curve, mph;
- t_n = Deceleration time without brakes (assumed to be 3 s), s;
- d_n = Deceleration rate without brakes, ft/s^2; and
- d_{wb} = Deceleration rate with brakes, ft/s^2.

Metric

$$D = 0.278V_h t_n - 0.5d_n(t_n)^2 + \frac{(0.278V_r)^2 - (0.278V_a)^2}{2d_{wb}} \qquad \textbf{(14-1, Metric)}$$

$$V_a = \frac{0.278V_h + d_n t_n}{0.278} \qquad \textbf{(14-2, Metric)}$$

Where:

V_h = Highway speed, km/h;
V_a = Speed after t_n seconds of deceleration without brakes, km/h;
V_r = Entering speed for controlling exit ramp curve, km/h;
t_n = Deceleration time without brakes (assumed to be 3 s), s;
d_n = Deceleration rate without brakes, m/s^2 and
d_{wb} = Deceleration rate with brakes, m/s^2.

The research team used the written values in the above equations, but reasonable approximations of the 1965 *Blue Book* values were not obtained. Distances were measured from the 1965 *Blue Book* Figure VII-15 and deceleration rates for selected speed changes were calculated. Using those deceleration rates and modifying slightly to improve the prediction provided a reasonable reproduction of the distances presented in the 1965 *Blue Book* Figure VII-16. Table 14-8 and Table 14-9 provide the calculated deceleration lengths along with the percent difference for each length for US customary and metric values, respectively. Overall, with the revised deceleration rates, the percent difference averages to zero. However, some individual lengths had as much as a 9 percent difference between the value in the *Blue Book* and the value calculated. Table 14-10 lists the deceleration rates used in reproducing the *Blue Book* values, while Figure 14-2 illustrates the deceleration rates.

Table 14-8. Reproduction of 1965 *Blue Book* Deceleration Length Values (US Customary) (*24*).

Highway Design Speed (mph)	Average Running Speed (mph)	Deceleration Length, D (ft) for Exit Curve Design Speed (mph)								
		Stop	15	20	25	30	35	40	45	50
		Assumed Exit Curve Speed (mph)								
		0	14	18	22	26	30	36	40	44
1965 *Blue Book* Figure VII-16: Derivation of Lengths (ft) for Deceleration Lane										
30	28	235	185	160	140	--	--	--	--	--
40	36	315	295	265	235	185	155	--	--	--
50	34	435	405	385	355	315	285	225	175	--
60	52	530	500	490	460	430	410	340	300	240
65	55	570	540	530	490	480	430	380	330	280
70	58	615	590	570	550	510	490	430	390	340
75	61	660	630	610	590	560	530	470	440	390
80	64	700	680	660	640	610	580	530	490	450
Calculated Deceleration Length (ft) Using Deceleration Rates Listed in Table 14-9										
30	28	232	190	162	127	--	--	--	--	--
40	36	329	291	266	235	198	155	--	--	--
50	34	432	398	375	347	314	275	205	--	--
60	52	540	509	488	462	432	396	332	284	230
65	55	581	551	531	506	476	442	380	333	281
70	58	623	594	574	550	521	488	428	382	332
75	61	665	637	618	595	566	534	476	431	382
80	64	708	680	662	639	612	580	524	481	433
Percent Difference (%) between Calculated and 1965 *Blue Book* Values										
30	28	1	-2	-1	9	--	--	--	--	--
40	36	-4	1	-1	0	-7	0	--	--	--
50	34	1	2	2	2	0	4	9	--	--
60	52	-2	-2	0	-1	0	3	2	5	4
65	55	-2	-2	0	-3	1	-3	0	-1	0
70	58	-1	-1	-1	0	-2	0	0	2	2
75	61	-1	-1	-1	-1	-1	-1	-1	2	2
80	64	-1	0	0	0	0	0	1	2	4
Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m										

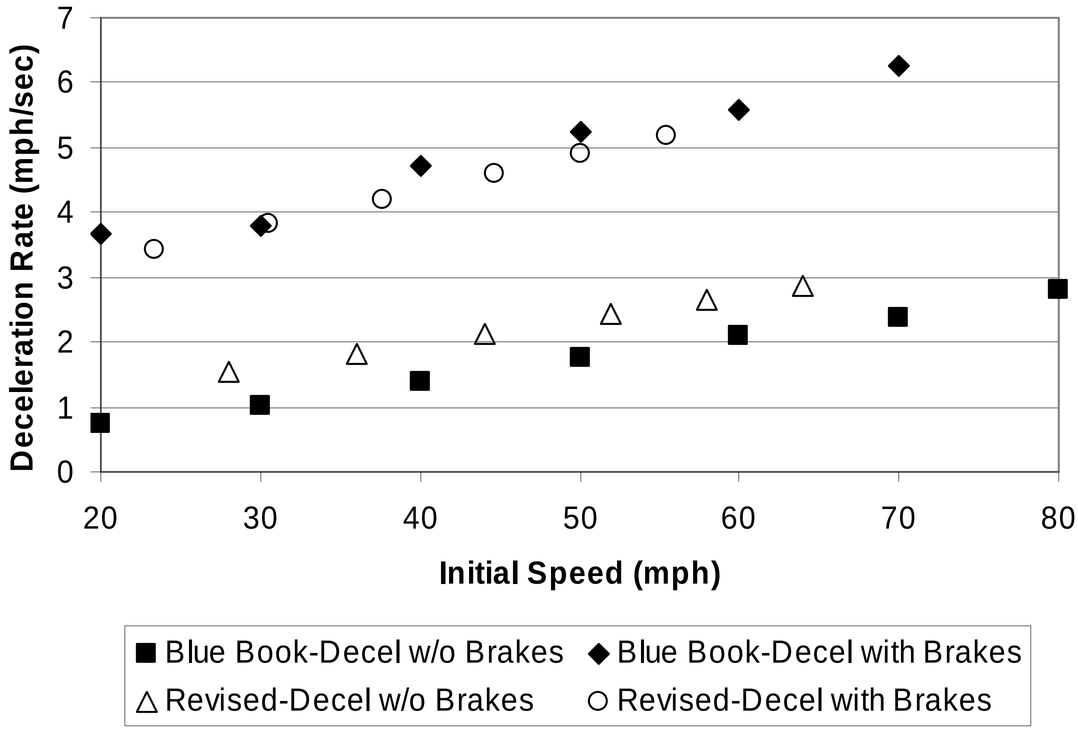
Table 14-9. Reproduction of 1965 *Blue Book* Deceleration Length Values (Metric) (*24*).

Highway Design Speed (km/h)	Average Running Speed (km/h)	Deceleration Length, D (m) for Exit Curve Design Speed (km/h)							
		Stop	**20**	**30**	**40**	**50**	**60**	**70**	**80**
		Assumed Exit Curve Speed (km/h)							
		0	**20**	**28**	**35**	**42**	**51**	**63**	**70**
TRDM/2004 *Green Book* Lengths (m) for Deceleration Lane									
50	47	75	70	60	45	--	--	--	--
60	55	95	90	80	65	55	--	--	--
70	63	110	105	95	85	70	55	--	--
80	70	130	125	115	100	90	80	55	--
90	77	145	140	135	120	110	100	75	60
100	85	170	165	155	145	135	120	100	85
110	91	180	180	170	160	150	140	120	105
120	98	200	195	185	175	170	155	140	120
Calculated Deceleration Length (m) Using Deceleration Rates Listed in Table 14-9									
50	47	74	64	54	43	--	--	--	--
60	55	92	83	74	64	51	--	--	--
70	63	112	103	94	85	73	55	--	--
80	70	129	120	112	103	92	75	46	--
90	77	146	138	131	122	111	95	68	49
100	85	167	159	152	144	134	118	93	75
110	91	182	175	168	160	150	135	111	94
120	98	200	194	187	179	170	156	132	116
Percent Difference (%) between Calculated and 2004 *Green Book* Values									
50	47	1	9	9	4	--	--	--	--
60	55	3	8	7	2	7	--	--	--
70	63	-1	2	1	0	-4	1	--	--
80	70	1	4	2	-3	-2	7	16	--
90	77	-1	1	3	-2	-1	5	10	18
100	85	2	3	2	1	1	2	7	11
110	91	-1	3	1	0	0	3	8	10
120	98	0	1	-1	-2	0	0	6	3

Table 14-10. Deceleration Rates[a] Used to Reproduce 1965 *Blue Book* Values.

Assumed 1st Deceleration[b]				
	(US Customary)		(Metric)	
Average Running Speed (mph)	**ft/s²**	**mph/s**	**Average Running Speed (km/h)**	**m/s²**
28	2.24	1.53	47	0.74
32	2.46	1.68	55	0.81
36	2.68	1.83	63	0.89
40	2.90	1.98	70	0.97
44	3.12	2.13	77	1.04
48	3.34	2.28	85	1.12
52	3.56	2.43	91	1.20
55	3.72	2.54	98	1.27
58	3.89	2.65		
61	4.05	2.76		
64	4.22	2.88		
Assumed 2nd Deceleration[c]				
	(US Customary)		**(Metric)**	
Speed after 3 s without Brakes (mph)	**ft/s²**	**mph/s**	**Speed after 3 s without Brakes (km/h)**	**m/s²**
23.4	5.01	3.42	39.1	1.54
27.0	5.30	3.61	46.2	1.65
30.5	5.59	3.81	53.4	1.76
34.1	5.88	4.01	59.6	1.86
37.6	6.17	4.20	65.7	1.96
41.2	6.45	4.40	72.9	2.07
44.7	6.74	4.60	78.1	2.15
47.4	6.96	4.75	84.2	2.24
50.1	7.18	4.89		
52.7	7.39	5.04		
55.4	7.61	5.19		

[a] Deceleration rates were estimated from 1965 *Blue Book* Figure VII-15 for initial speeds of 70 and 30 mph. The deceleration rates for 70 and 30 mph were modified to decrease the overall percent difference between the calculated values and the *Blue Book* values. The other speeds were determined as a proportion of the deceleration rate difference between 70 and 30 mph (i.e., assumed there was a linear relationship of deceleration rate to speed). Figure 14-2 illustrates the deceleration rates. The deceleration rates for metric were a conversion from the US customary values for initial speeds of 70 and 30 mph and then determined as a proportion of the deceleration rate differences between 48 and 161 km/h.
[b] Accelerator pedal is released and the vehicle slows in gear without the use of brakes.
[c] Brakes are applied.



Note: 1 mph = 1.61 km/h

Figure 14-2. Deceleration Rates Estimated from *Blue Book* Figure VII-15 and Revised to Improve Calculations of Deceleration Lengths.

Deceleration and Acceleration Taper Lengths

Minimum taper lengths from the TxDOT *Roadway Design Manual* are provided in Tables 14-1 and 14-3. These values are similar but not exactly the same as the values in the 1965 *Blue Book* (see Table 14-11). The same dimensions are provided for either an acceleration lane or a deceleration lane. The 2004 *Green Book* does not include the minimum taper length values listed in the TRDM; rather, it has the following note for acceleration lengths: "uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1300 ft [400 m]" and does not provide written guidance in the deceleration length table (see *Green Book* Exhibit 10-73) (2). Guidance on taper length is provided within some of the *Green Book* figures; for example, GB Exhibit 10-72 shows 250 ft [75 m] taper length for a parallel design.

Because the *Blue Book* did not include discussion on how to calculate the taper lengths, various techniques were used to try to reproduce the values. Part of the procedure to develop the deceleration lengths in the *Blue Book* is to determine the distance traveled while decelerating without brakes. That distance is similar to the length of taper but is about 15 to 30 percent less than the values in the TRDM. Hunter and Machemehl *(56)* reported taper lengths were based on passing practices on two-lane highways as determined in a 1941 study *(57)*. The 1941 study found depending on traffic conditions a passing vehicle would shift laterally one lane in 2.6 to

4.1 seconds. From this it was assumed that the time required for a driver to shift from a through lane to a speed-change lane was 3 seconds minimum to 4 seconds desirable, leading to a value of 3.5 seconds for design. The taper length is calculated as 3.5 s times the average running speed in ft/s. Figure 14-3 illustrates the various values and Table 14-11 lists the values.

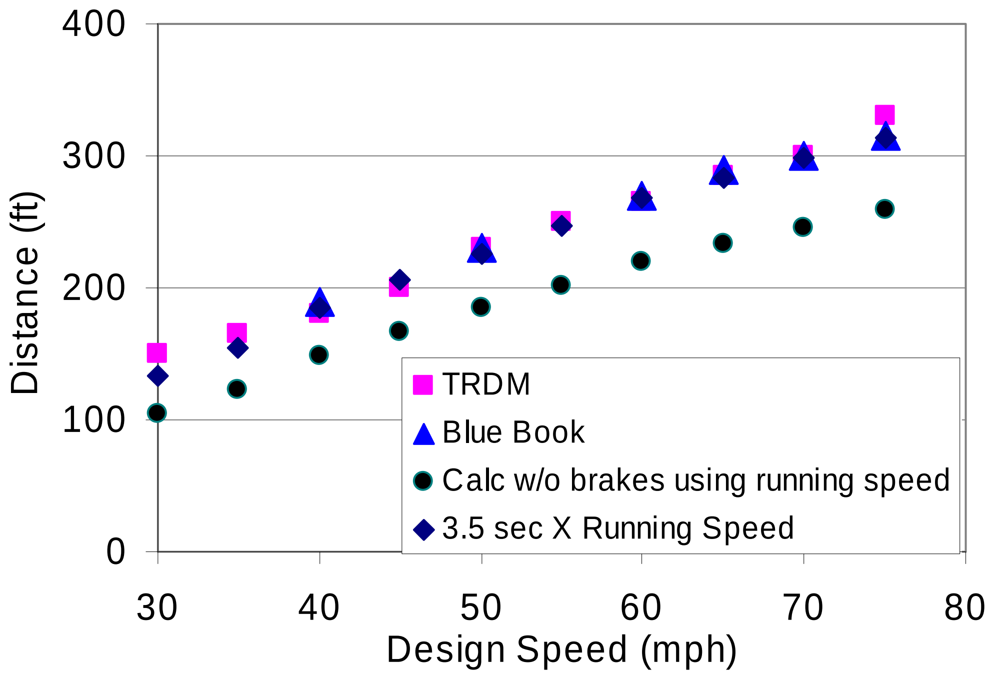
Table 14-11. Comparison of Taper Lengths.

(US Customary)

Design Speed (mph)	Average Running Speed (mph)	Length of Taper (ft)		Difference, *Blue Book* – TRDM (mph)	Distance during Deceleration without Brakes (mph)	Length Based on Average Running Speed in ft/s × 3.5 s (ft)
		TRDM	*Blue Book*			
30	26	150	--	--	105	134
35	30	165	--	--	123	154
40	36	180	190	-10	149	185
45	40	200	--	--	167	206
50	44	230	230	0	184	226
55	48	250	--	--	202	247
60	52	265	270	-5	220	268
65	55	285	290	-5	233	283
70	58	300	300	0	246	298
75	61	330	315	15	259	314
80	64	--	330	--	273	329

(Metric)

Design Speed (km/h)	Average Running Speed (km/h)	Length of Taper (m)		Difference, Blue Book – TRDM (km/h)	Distance during Deceleration without Brakes (km/h)	Length Based on Average Running Speed in m/s × 3.5 s (m)
		TRDM	Blue Book			
50	47	45	--	--	36	46
60	55	55	--	--	42	54
70	63	60	--	--	49	61
80	70	70	--	--	54	68
90	77	75	--	--	60	75
110	85	80	--	--	66	83
110	91	90	--	--	71	89
120	98	100	--	--	76	95



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 14-3. Taper Lengths.

Acceleration Lengths

Similar to deceleration lengths, acceleration length values are included in the *Policies on Geometric Highway Design* published by AASHO in 1954 (*4*) and the 1965 *Blue Book* (*24*). The 1954 *Policies* values are not as close to the TRDM values as the 1965 *Blue Book* values. Therefore, comparisons will be made to the 1965 *Blue Book* values. The observation that taper length was included as part of the 1965 *Blue Book* acceleration length and is now not included in the TRDM or 2004 *Green Book* is also valid for acceleration lengths. The TRDM values match the values in the 2004 *Green Book*. For some combinations, the TRDM (and the 2004 *Green Book*) acceleration lengths differ slightly from the 1965 *Blue Book* values. They also include values for 35-, 45-, 55-, 65-, and 75-mph [56, 72, 89, 105, 121 km/h] design speeds that were not included in the 1965 *Blue Book*. None of the documents included values for an 80-mph [129 km/h] highway design speed. The differences between the TRDM/2004 *Green Book* and the 1965 *Blue Book* values are listed in Table 14-12. The largest difference was for a 50-mph [81 km/h] highway design speed from a 30-mph [48 km/h] entrance curve design speed. The TRDM and the 2004 *Green Book* have a 450-ft [137 m] acceleration length while the 1965 *Blue Book* has a 500-ft [153 m] dimension – a difference of 50 ft [15 m].

Table 14-12. Lengths of Entrance Ramp Speed Change Lanes from TRDM/2004 *Green Book* and 1965 *Blue Book* (1, 2, 24).

Highway Design Speed (mph)	Difference in Acceleration Length (ft) between TRDM/2004 *Green* Book and 1965 *Blue* Book for Entrance Curve Design Speed (mph)								
	Stop	15	20	25	30	35	40	45	50
	Assumed Initial Speed (mph)								
	0	14	18	22	26	30	36	40	44
30	-10	NB	--	--	--	--	--	--	--
35	NB	NB	NB	NB	NB	--	--	--	--
40	-20	-20	20	-10	-20	--	--	--	--
45	NB	NB	NB	NB	NB	NB	--	--	--
50	-40	-40	-20	-30	-50	-30	-30	-30	--
55	NB	NB	NB	NB	NB	NB	NB	NB	--
60	30	20	30	20	0	0	-40	20	10
65	NB	NB	NB	NB	NB	NB	NB	NB	NB
70	30	20	20	10	20	0	-10	-10	0
75	NB	NB	NB	NB	NB	NB	NB	NB	NB
80	NT	NT	NT	NT	NT	NT	NT	NT	NT

NB = value not included in 1965 *Blue Book*.
NT = value not included in TRDM.
Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

The 1965 *Blue Book* states that the length of the acceleration lane is based on the following factors in combination (24):

A. the speed at which drivers enter the acceleration lane (i.e., the speed at the end of the ramp's controlling curve);
B. the manner of accelerating or the acceleration factors; and
C. the speed at which drivers merge with through traffic (or stated in another manner, the speed at the end of the acceleration lane).

For factor A, the 1965 *Blue Book* uses average running speeds, similar to the values used to calculate deceleration lengths (see Table 14-7). The assumption is that drivers will exit the curve at a speed (called the average running speed) that is less than the curve's design speed. This conservative assumption is not currently supported by research findings or operational experience.

For factor B, the 1965 *Blue Book* provides graphs that show acceleration rates for different conditions. The curve used to generate the 1965 *Blue Book* acceleration rates was for "normal acceleration" for passenger vehicles on level grade as determined in a 1937 Bureau of Public Roads study. Per the 1965 *Blue Book,* the data were converted to show "distance traveled while accelerating from one to another." The resulting curves could be used to determine the acceleration distance from initial speed values to speed reached values, although the 1965 *Blue Book* also included the tabulated values.

For factor C, the 1965 *Blue Book* states that the speed of the entering vehicles should approximate that of the through traffic that would be equal to the average running speed of traffic on the highway. Later in the same section, the 1965 *Blue Book* states "it is satisfactory and does not unduly inconvenience through traffic for vehicles from the acceleration lane to enter the through pavement at a speed approximately 5 mph less." The 1965 *Blue Book* lengths were determined on that basis (initial speed as the average running speed of the turning roadway curve and speed reached is 5 mph [8 km/h] less than the average running speed of through traffic).

The 1965 *Blue Book* provided the following observations with respect to trucks and buses:

> "Lengths of acceleration lane are based on passenger vehicle operation. Trucks and buses generally require much longer distances to accelerate, and lengths based thereon would be entirely out of reason. A slower entry of trucks and buses is unavoidable and generally accepted by the traveling public. Where a substantial number of large vehicles are to enter a high speed highway, the length should be increased, or the entry located on a downgrade if feasible" (*24*).

The caution expressed in the deceleration length section of this chapter also applies to the acceleration length calculations – drivers do not routinely drive less than the design speed of the facility.

Using the information provided in the 1965 *Blue Book* within the uniform acceleration formula (i.e., distance traveled is the difference between the square of the speeds divided by two times the acceleration), reasonable approximations were obtained. The following equation was used to calculate acceleration lengths:

US Customary

$$A = \frac{(1.47V_h)^2 - (1.47V_r)^2}{2a}$$

Metric

$$A = \frac{(0.278V_h)^2 - (0.278V_r)^2}{2a} \qquad \textbf{(14-3)}$$

Where:
- A = Acceleration length, ft;
- V_h = Highway speed, mph;
- V_r = Speed on controlling curve for ramp, mph; and
- a = Acceleration rate, ft/s^2.

Where:
- A = Acceleration length, m;
- V_h = Highway speed, km/h;
- V_r = Speed on controlling curve for ramp, km/h; and
- a = Acceleration rate, m/s^2.

Table 14-13 lists the values calculated. A similar approach was used to regenerate the values included in the TRDM and 2004 *Green Book* because these values were as much as 50 ft [15 m] different from the *Blue Book* values. Some of the acceleration rates precisely generated the desired acceleration lengths, but the rates did not necessarily follow a logical pattern. This finding was similar to the findings from reproducing the 1965 *Blue Book* values. Overall the pattern is a decreasing acceleration rate as the initial speed increases. A review of the rates reveals isolated situations when that overall pattern is not followed. For example, in Tables 14-14 and 14-15, the rate for a vehicle moving from 18 to 31 mph [29 to 50 km/h] is a higher rate (2.75 ft/s^2 [0.84 m/s^2]) than the rate calculated to reproduce the acceleration length for a vehicle moving from 14 to 31 mph [23 to 50 km/h] (2.58 ft/s^2 [0.79 m/s^2]). The decision was made to identify a logical pattern in the acceleration rates rather than reproducing the exact numbers because the TRDM/2004 *Green Book* values may have been rounded or adjusted over the years. Following the logical pattern would provide a basis for extrapolating the acceleration rates into the higher speeds. Table 14-14 and Table 14-15 list the reproduced TRDM/2004 *Green Book* values along with the acceleration rates used to generate those values in US customary and metric units, respectively. Figure 14-4 shows a plot of the acceleration rates for highway design speeds of 30, 40, 50, 60, and 70 mph [48, 64, 81, 97, and 113 km/h] (speed reached of 23, 31, 39, 47, and 53 mph [37, 50, 63, 76, and 85 km/h]). Because the *Blue Book* did not include acceleration lane length for a design speed of 75 mph [121 km/h], it was assumed that the same rates for 70 mph [113 km/h] would apply for a 75-mph [121 km/h] highway design speed.

Table 14-13. Reproduction of 1965 *Blue Book* Acceleration Length Values (*24*).

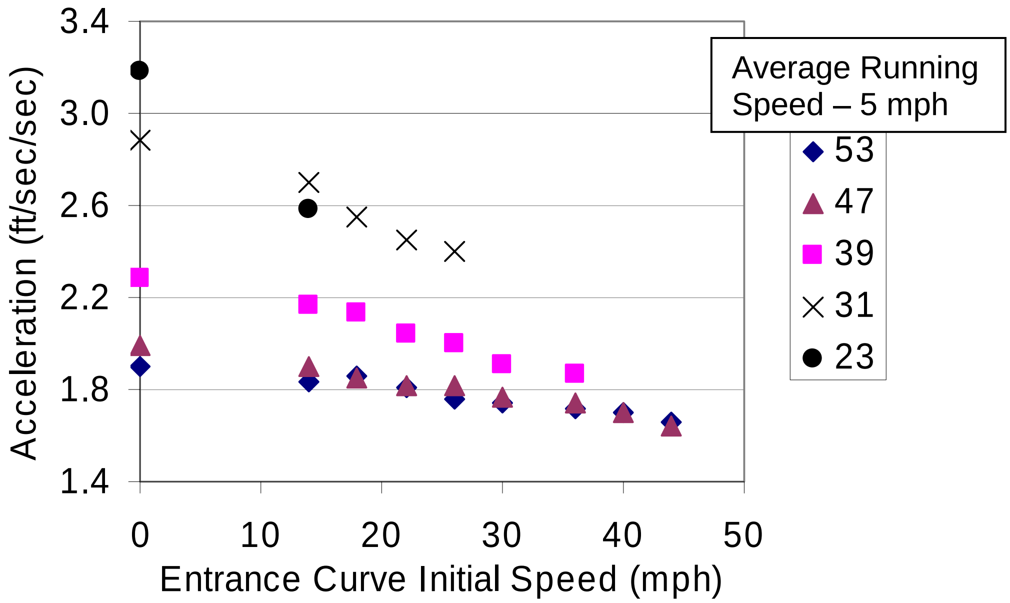
Highway Design Speed (mph)	**Average Running Speed-5 (mph)**	**Acceleration Length, A (ft) for Entrance Curve Design Speed (mph)**								
		Stop	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
		Initial Speed (mph)								
		0	**14**	**18**	**22**	**26**	**30**	**36**	**40**	**44**
1965 *Blue Book* Figure VII-18: Derivation of Lengths (ft) for Acceleration Lane										
30	23	190	--	--	--	--	--	--	--	--
40	31	380	320	250	220	140	--	--	--	--
50	39	760	700	630	580	500	380	160	--	--
60	47	1170	1120	1070	1000	910	800	590	400	170
70	53	1590	1540	1500	1410	1330	1230	1010	830	580
Calculated Acceleration Length (ft) Using Acceleration Rates Listed Below										
30	23	191	--	--	--	--	--	--	--	--
40	31	378	320	250	219	140	--	--	--	--
50	39	757	700	632	578	502	381	160	--	--
60	47	1164	1115	1066	997	910	799	587	399	170
70	53	1597	1543	1500	1403	1324	1228	1009	827	582
Acceleration Rates (ft/s^2) Used to Reproduce Acceleration Lengths										
30	23	3.00	--	--	--	--	--	--	--	--
40	31	2.75	2.58	2.75	2.35	2.20	--	--	--	--
50	39	2.17	2.05	2.05	1.94	1.82	1.76	1.52	--	--
60	47	2.05	1.95	1.91	1.87	1.82	1.77	1.68	1.65	1.73
70	53	1.90	1.83	1.79	1.79	1.74	1.68	1.62	1.58	1.62
Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m										

Table 14-14. Reproduction of TRDM and 2004 *Green Book* Acceleration Length Values (US Customary) (1, 2).

Highway Design Speed (mph)	Average Running Speed-5 (mph)	Acceleration Length, A (ft) for Entrance Curve Design Speed (mph)								
		Stop	15	20	25	30	35	40	45	50
		Initial Speed (mph)								
		0	14	18	22	26	30	36	40	44
TRDM										
30	23	180	140	--	--	--	--	--	--	--
35	27	280	220	160	--	--	--	--	--	--
40	31	360	300	270	210	120	--	--	--	--
45	35	560	490	440	380	280	160	--	--	--
50	39	720	660	610	550	450	350	130	--	--
55	43	960	900	810	780	670	550	320	150	--
60	47	1200	1140	1100	1020	910	800	550	420	180
65	50	1410	1350	1310	1220	1120	1000	770	600	370
70	53	1620	1560	1520	1420	1350	1230	1000	820	580
75	56	1790	1730	1630	1580	1510	1420	1160	1040	780
Calculated Acceleration Length (ft) Using Acceleration Rates Listed Below										
30	23	180	139	--	--	--	--	--	--	--
35	27	260	218	162	--	--	--	--	--	--
40	31	361	306	270	210	128	--	--	--	--
45	35	513	457	416	357	270	160	--	--	--
50	39	721	660	607	549	456	351	130	--	--
55	43	936	878	828	764	664	557	331	149	--
60	47	1199	1145	1101	1024	910	799	567	387	180
65	50	1389	1335	1267	1200	1101	985	752	572	369
70	53	1597	1543	1444	1388	1309	1185	950	768	568
75	56	1783	1736	1633	1583	1510	1388	1156	976	772
Acceleration Rates (ft/s^2) Used to Reproduce Acceleration Lengths										
30	23	3.18	2.58	--	--	--	--	--	--	--
35	27	3.03	2.64	2.70	--	--	--	--	--	--
40	31	2.88	2.70	2.55	2.45	2.40	--	--	--	--
45	35	2.58	2.44	2.34	2.25	2.20	2.20	--	--	--
50	39	2.28	2.17	2.13	2.04	2.00	1.91	--	--	--
55	43	2.14	2.04	1.99	1.93	1.91	1.84	1.80	--	--
60	47	1.99	1.90	1.85	1.82	1.82	1.77	1.70	1.64	--
65	50	1.95	1.87	1.86	1.82	1.79	1.76	1.70	1.65	1.60
70	53	1.90	1.83	1.86	1.81	1.76	1.74	1.70	1.66	1.65
75	56	1.90	1.83	1.86	1.81	1.76	1.74	1.70	1.68	1.65

Table 14-15. Reproduction of TRDM and 2004 *Green Book* Acceleration Length Values (Metric) (*1*, *2*).

Highway Design Speed (km/h)	Average Running Speed-10 (km/h)	Acceleration Length, A (m) for Entrance Curve Design Speed (km/h)							
		Stop	**20**	**30**	**40**	**50**	**60**	**70**	**80**
		Initial Speed (km/h)							
		0	**20**	**28**	**35**	**42**	**51**	**63**	**70**
TRDM									
50	37	60	50	30	--	--	--	--	--
90	45	95	80	65	45	--	--	--	--
70	53	150	130	110	90	65	--	--	--
80	60	200	180	165	145	115	65	--	--
90	67	260	245	225	205	175	125	35	--
100	75	345	325	305	285	255	205	110	40
110	81	430	410	390	370	340	290	200	125
120	88	545	530	515	490	460	410	325	245
Calculated Acceleration Length (m) Using Acceleration Rates Listed Below									
50	37	60	50	30	--	--	--	--	--
90	45	95	81	65	44	--	--	--	--
70	53	150	129	109	90	64	--	--	--
80	60	199	177	165	143	113	64	--	--
90	67	259	243	224	200	170	122	33	--
100	75	345	326	302	283	249	201	110	40
110	81	438	427	394	374	346	289	191	124
120	88	544	526	507	484	453	397	317	244
Acceleration Rates (m/s^2) Used to Reproduce Acceleration Lengths									
50	37	0.88	0.75	0.75	--	--	--	--	--
90	45	0.82	0.78	0.74	0.70	--	--	--	--
70	53	0.73	0.72	0.72	0.68	0.63	--	--	--
80	60	0.70	0.70	0.66	0.64	0.63	0.60	--	--
90	67	0.67	0.65	0.64	0.63	0.62	0.60	0.60	--
100	75	0.63	0.62	0.62	0.60	0.60	0.58	0.58	0.70
110	81	0.58	0.56	0.57	0.55	0.54	0.53	0.52	0.52
120	88	0.55	0.54	0.53	0.52	0.51	0.50	0.46	0.45



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 14-4. US Customary Acceleration Rates Used to Generate Acceleration Lengths.

Adjustment Factors

The TRDM includes adjustment factors for deceleration or acceleration on a ramp with a grade of 3 to 4 percent or 5 to 6 percent. Adjustment factors were also included in the 1965 *Blue Book* (*Blue Book* Table VII-11), and given the similarities between the values it appears that the source of the TRDM could be the values in the 1965 *Blue Book*. The values presented in the 1965 *Blue Book* were for fewer design speed increments than currently provided in the TRDM and the 2004 *Green Book*. For example, the TRDM provides values on 5-mph [8 km/h] increments while the *Blue Book* only provides values on 10-mph [16 km/h] increments. The added 5-mph [8 km/h] increments appear to be an average of the neighboring adjustment factors. For example, the adjustment factor for a 30-mph [48 km/h] turning roadway curve on a 3 to 4 percent upgrade for 40- and 50-mph [64 and 81 km/h] design speeds are 1.3 and 1.4, respectively. The adjustment factor for 45 mph [72 km/h] in the TRDM is 1.35, which could be the average of the 1.3 and 1.4 values. The source of the adjustment factors was not provided in the 1965 *Blue Book*, and no supporting discussions for the adjustment factors were included.

The source of the adjustment factors per the 1954 *Blue Book* (4) was to apply principles of mechanics to rates of speed change for level grades. Stated in another way, engineering judgment was used to determine the adjustment factors. The direct quote from the 1954 *Blue Book* follows:

> "Deceleration distances are longer on downgrades and shorter on upgrades, while acceleration distances are longer on upgrades and shorter on downgrades. Data on driver behavior while decelerating or accelerating on grades are not available, but they may be approximated by applying principles of mechanics to rates of

speed change for level grades, recognizing that drivers accelerating on upgrades open throttles more than the equivalent for normal acceleration on level grades. Calculations result in lengths of acceleration and deceleration lanes on grades as compared with those on the level as summarized in Table VII-11 {see Table 14-2 and 14-4 in this report}. The ratio from this table multiplied by the length in Table VII-10 gives the length of speed-change lane on grade" (*4*).

POTENTIAL VALUES FOR HIGH DESIGN SPEEDS

Deceleration Lengths

A procedure for calculating the deceleration lengths currently in the TRDM was identified (see Discussion section above). This procedure was used to calculate the deceleration lengths for the higher speeds (see Table 14-16 for US customary units and Table 14-17 for metric units). Figure 14-5 illustrates a sample of the calculated deceleration lengths.

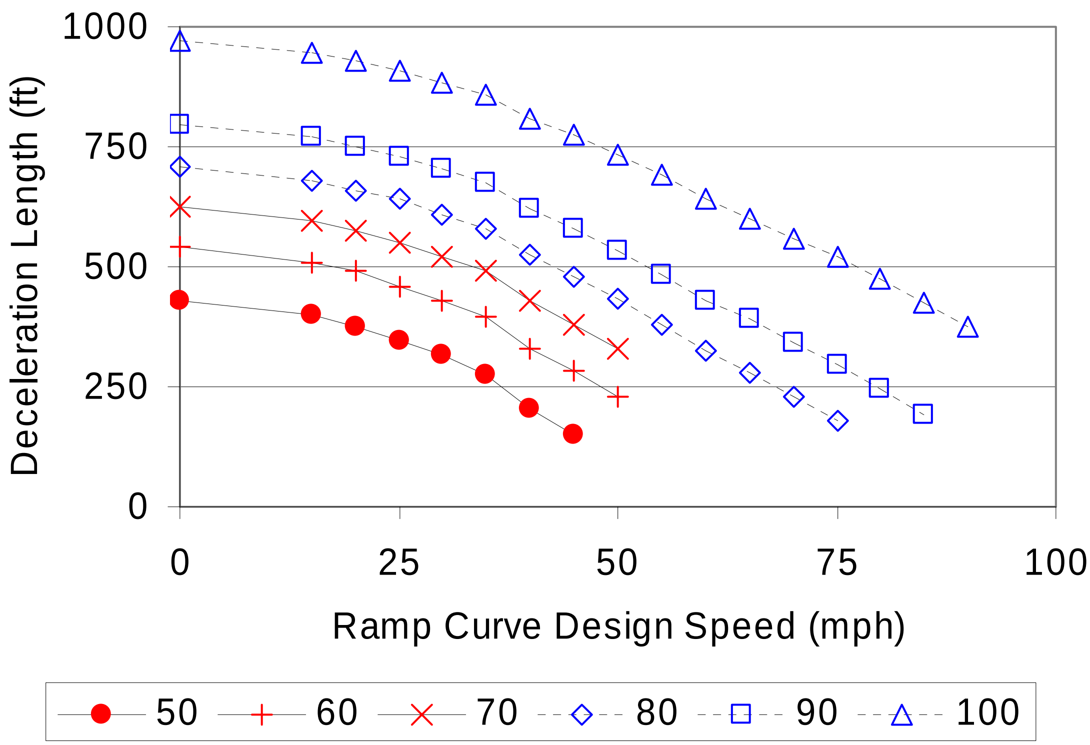
While a procedure was identified, and that procedure can be used to calculate the deceleration lengths for higher speeds, several assumptions used in the procedure may require testing and revision. The research team's initial thoughts on those assumptions are provided in the Research Needs section of this chapter. Note that a current NCHRP project (15-31) (*52*) is investigating ramp design, and the findings from that research could result in changes to the procedure for calculating design deceleration lengths. When the results of the NCHRP project are available, or when additional investigations can be performed, revised deceleration lengths may be identified.

Table 14-16. Potential Deceleration Lengths (US Customary).

(US Customary)															
Highway Design Speed (mph)	**Average Running Speed (mph)**	**Deceleration Length, D (ft) for Exit Curve Design Speed (mph)**													
		Stop	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**	**75**
		Assumed Exit Curve Speed (mph)													
		0	**14**	**18**	**22**	**26**	**30**	**36**	**40**	**44**	**48**	**52**	**55**	**58**	**61**
30	28	**Existing Criteria in *Roadway Design Manual* Figure 3-36**									--	--	--	--	--
35	32										--	--	--	--	--
40	36										--	--	--	--	--
45	40										--	--	--	--	--
50	44										--	--	--	--	--
55	48										--	--	--	--	--
60	52										185	--	--	--	--
65	55										225	185	--	--	--
70	58										270	225	190	--	--
75	61										310	265	235	195	--
80	64	605	580	570	550	530	505	465	430	395	355	310	275	240	200
85	67	650	630	615	595	575	550	510	475	440	400	355	325	285	250
90	70	695	675	660	645	625	600	555	525	490	450	405	370	335	295
95	75	780	760	745	725	705	680	640	605	570	530	485	455	415	375
100	82	900	880	865	850	830	805	760	730	695	655	610	575	540	500
Shaded areas reflect high-design-speed potential values.															

Table 14-17. Potential Deceleration Lengths (Metric).

(Metric)													
Highway Design Speed (km/h)	**Average Running Speed (km/h)**	**Deceleration Length, D (m) for Exit Curve Design Speed (km/h)**											
		Stop	**20**	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**	**110**	**120**
		Assumed Exit Curve Speed (km/h)											
		0	**20**	**28**	**35**	**42**	**51**	**63**	**70**	**77**	**85**	**91**	**98**
50	47	**Existing Criteria in *Roadway Design Manual* Figure 3-36**								--	--	--	--
60	55									--	--	--	--
70	63									--	--	--	--
80	70									--	--	--	--
90	77									--	--	--	--
100	85									56	--	--	--
110	91									78	52	--	--
120	98									102	78	58	--
130	102	225	217	210	202	192	177	152	135	116	92	73	--
140	110	248	241	234	226	217	202	178	162	144	121	103	80
150	118	271	264	258	250	241	227	204	189	172	150	132	110
160	131	309	303	297	290	282	268	248	233	216	196	180	159
Shaded areas reflect high-design-speed potential values.													



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 14-5. Deceleration Lengths for a Sample of Speeds.

Taper Lengths

Taper lengths for the higher design speeds were determined as 3.5 s multiplied by the average running speed (in ft/s). Table 14-18 lists the results. A taper length for an 80-mph [130 km/h] design speed is also provided because this value is not currently in the TRDM, as well as for 75-mph [120 km/h] design speed because the calculated value for 80 mph [130 km/h] matches the current value for 75 mph [120 km/h].

Hunter and Machemehl *(56)* reported on a 1989 NCHRP study that found the following distribution of times for a driver to steer from the acceleration lane completely onto the adjacent freeway lane: 1.25 s (15th percentile), 1.75 s (50th percentile), and 3.24 s (85th percentile). The 85th percentile value of 3.24 s is near the value of 3.5 s determined based on the 1941 study *(57)*.

Table 14-18. Potential Taper Lengths.

(US Customary)			
Design Speed (mph)	**Average Running Speed (mph)**	**Length of Taper (ft)**	
		Calculated [a]	**Rounded**
75	61	314	315
80	64	329	330
85	67	345	345
90	70	360	360
95	75	386	370
100	82	422	425
(Metric)			
Design Speed (km/h)	**Average Running Speed (km/h)**	**Length of Taper (m)**	
		Calculated [b]	**Rounded**
120	98	95	95
130	102	99	100
140	110	107	110
150	118	115	115
160	131	127	130

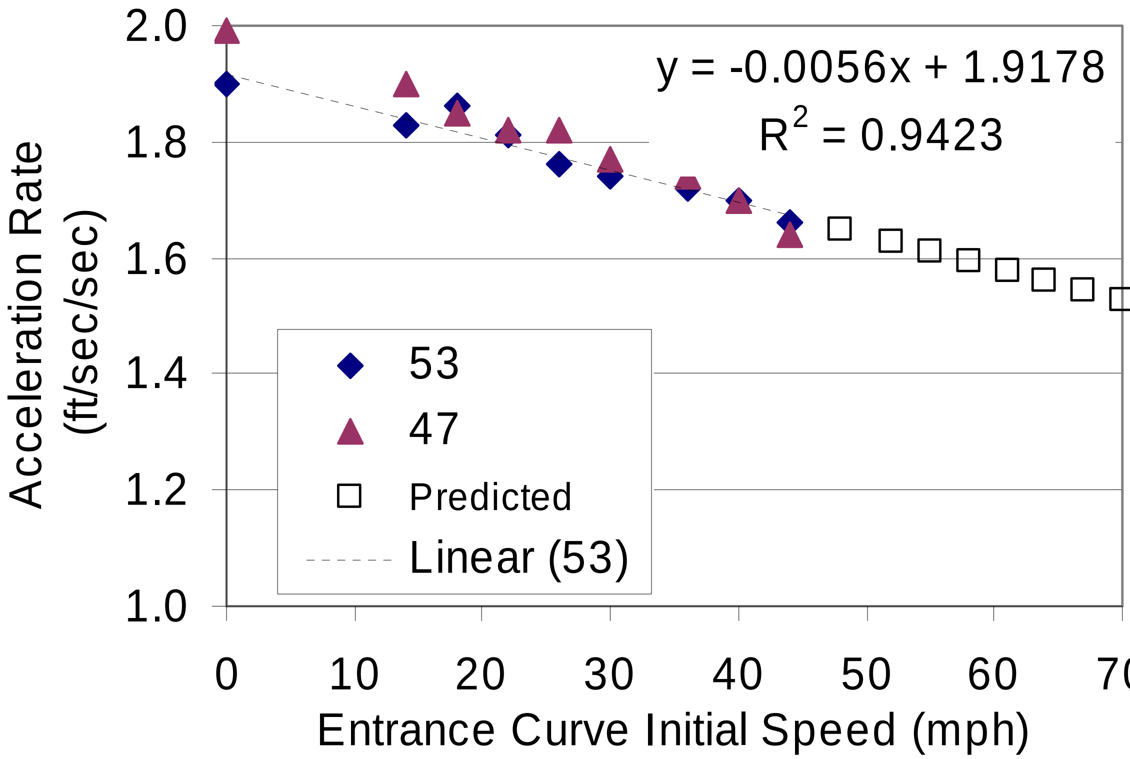
[a] Determined using following equation: Taper Length = 3.5 × 1.47 × (average running speed in ft/s).
[b] Determined using following equation: Taper Length = 3.5 × 0.278 × (average running speed in m/s).
Shaded areas reflect high-design-speed potential values.

Acceleration Lengths

A procedure for calculating the acceleration lengths currently in the TRDM was identified (see Discussion section above). Similar concerns with the dated assumptions used in the deceleration lengths calculations exist with the acceleration calculations. Additional investigations are needed. Until such research can be performed the following assumptions or decisions were made to generate the acceleration lengths:

- Average running speed relationships as developed in other procedures (see Chapter 8) were used including the assumption that merging vehicles will be at 5 mph [8 km/h] less than the average running speeds.
- The acceleration rates assumed for the 70-mph [113 km/h] highway design speed was used for the higher highway speeds.
- A trend line was developed using the acceleration rates estimated for a 70-mph [113 km/h] highway design speed (speed reached of 53 mph [85 km/h]). The trend line was used to predict the acceleration rates for initial speeds of 48-, 52-, 55-, 58-, 61-, 64-, 67-, and 70-mph [77, 84, 89, 93, 98, 103, 108, and 113 km/h] entrance ramp curve operating speeds (design speeds of 55, 60, 65, 70, 75, 80, 85, and 90 mph [89, 97, 105, 113, 121, 129, 137, and 145 km/h]). Figure 14-6 shows the data and trend line.

The acceleration lengths for the higher design speeds are listed in Tables 14-19. Figure 14-7 shows a sample of acceleration lengths.



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 14-6. Acceleration Rate for Higher Speeds Determined Based on Using a Trend Line Developed from Acceleration Rates for Highway Average Running Speed of 53 mph [85 km/h].

Table 14-19. Potential Acceleration Lengths.

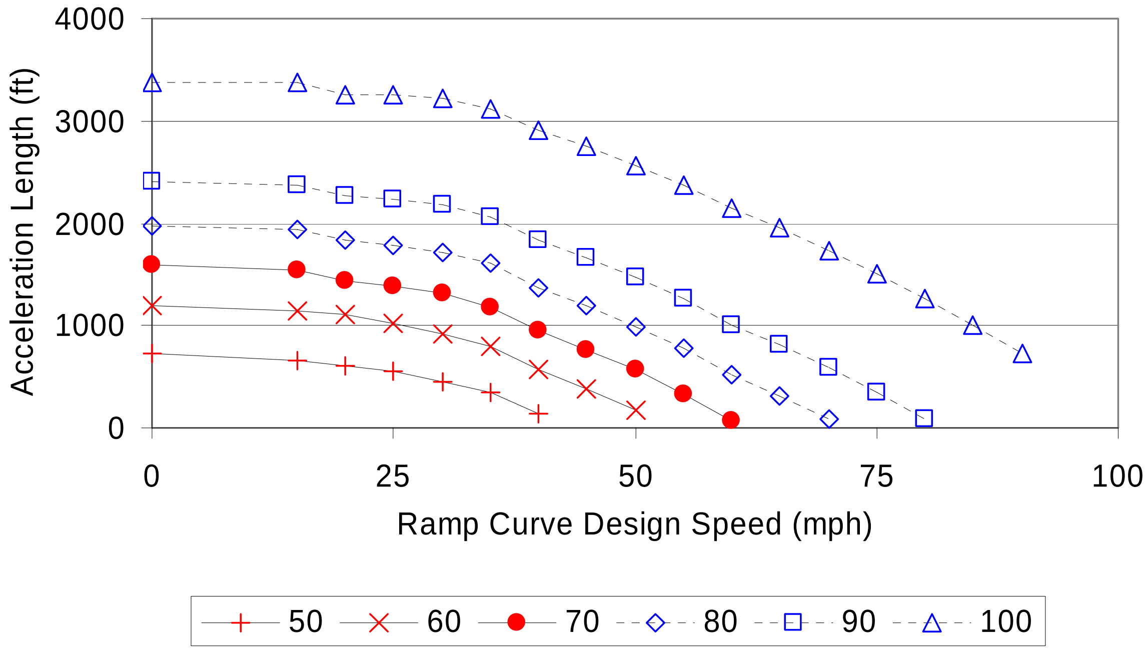
(US Customary)

HDS	ARS -5	Stop	15	20	25	30	35	40	45	50	55	60	65	70	75
		Acceleration Length, A (ft) for Entrance Curve Design Speed (mph)													
		Initial Speed (mph)													
		0	**14**	**18**	**22**	**26**	**30**	**36**	**40**	**44**	**48**	**52**	**55**	**58**	**61**
30	23										--	--	--	--	--
35	27										--	--	--	--	--
40	31										--	--	--	--	--
45	35										--	--	--	--	--
50	39	**Existing Criteria in *Roadway Design Manual* Figure 3-36**									--	--	--	--	--
55	43										--	--	--	--	--
60	47										--	--	--	--	--
65	50										132	--	--	--	--
70	53										331	70	--	--	--
75	56										545	287	74	--	--
80	59	1979	1939	1834	1789	1722	1603	1373	1195	994	771	516	306	79	--
85	62	2186	2154	2045	2006	1945	1828	1601	1426	1227	1009	757	550	326	84
90	65	2403	2379	2266	2233	2179	2065	1840	1668	1472	1259	1010	805	584	345
95	70	2786	2777	2658	2636	2593	2484	2264	2097	1906	1701	1459	1258	1042	808
100	77	3372	3385	3256	3250	3225	3123	2910	2751	2568	2375	2142	1949	1740	1514

(Metric)

HDS	ARS -10	Stop	20	30	40	50	60	70	80	90	100	110	120	--	--
		Acceleration Length, A (m) for Entrance Curve Design Speed (km/h)													
		Initial Speed (km/h)													
		0	**20**	**30**	**40**	**47**	**55**	**63**	**70**	**77**	**85**	**91**	**98**	**--**	**--**
50	37									--	--	--	--	--	--
60	45									--	--	--	--	--	--
70	53									--	--	--	--	--	--
80	60	**Existing Criteria in *Roadway Design Manual* Figure 3-36**								--	--	--	--	--	--
90	67									--	--	--	--	--	--
100	75									--	--	--	--	--	--
110	81									48	--	--	--	--	--
120	88									156	46	--	--	--	--
130	92	595	577	560	538	508	453	378	306	218	109	--	--	--	--
140	100	703	687	672	652	624	572	507	438	350	245	155	37	--	--
150	108	819	806	793	776	750	700	646	581	492	392	305	190	--	--
160	121	977	987	945	940	928	877	787	726	657	570	496	397	--	--

HDS = Highway Design Speed (mph or km/h).
ARS-5 = Average Running Speed-5 (mph).
ARS-10 = Average Running Speed -10 (km/h).
Shaded areas reflect high-design-speed potential values.



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 14-7. Acceleration Lengths for a Sample of Speeds.

Adjustment Factors

Because a logical procedure for calculating the adjustment factors has not yet been determined, trend lines for each series of adjustment factors were determined and used to extrapolate the factors into the higher design speeds. Figure 14-8 shows the plot of adjustment factors for 3 to 4 percent grades along with the trend line equation for the 45-mph [72 km/h] turning roadway curve design speed on an upgrade and the trend line for the downgrade adjustment factors. These trend lines (along with the others generated for each grade/ramp design speed combination) were used to predict the adjustment factor for the higher highway design speeds. The potential adjustment factors for acceleration lanes are listed in Table 14-20. Using this approach results in some very large adjustment factors as can be more easily seen in Figure 14-9 (3 to 4 percent grades) and Figure 14-10 (5 to 6 percent grades). Additional research is needed to determine if these adjustment factors are reasonable.

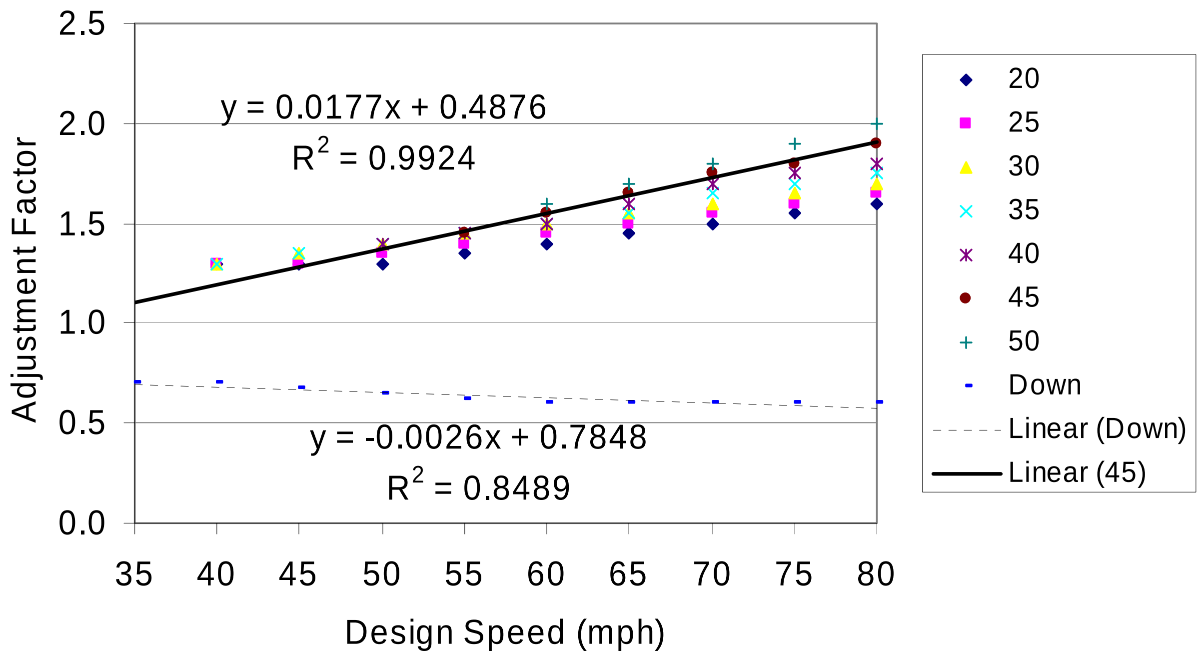
The adjustment factors for deceleration lanes are currently independent of design speed (see Table 14-2). Existing information does not indicate that adjustment factors for deceleration lanes should be based on design speed, although information on this topic is limited.

Table 14-20. Potential Speed Change Lane Adjustment Factors as a Function of a Grade for Acceleration Lanes.

(US Customary)								
Design Speed of Roadway (mph)	Ratio of Length on Grade to Length on Level [a]							
	20	25	30	35	40	45	50	All Speeds
	3 to 4% Upgrade							3 to 4% Downgrade
85	1.62	1.69	1.75	1.80	1.89	1.99	2.10	0.56
90	1.66	1.73	1.80	1.86	1.96	2.08	2.20	0.55
95	1.71	1.78	1.85	1.92	2.03	2.17	2.30	0.54
100	1.75	1.83	1.90	1.98	2.10	2.26	2.40	0.52
	5 to 6 % Upgrade							5 to 6 % Downgrade
85	2.39	2.51	2.64	2.94	3.15	3.73	4.28	0.46
90	2.50	2.64	2.77	3.10	3.33	4.00	4.65	0.45
95	2.62	2.76	2.91	3.27	3.51	4.26	5.03	0.44
100	2.74	2.89	3.04	3.43	3.69	4.53	5.40	0.42

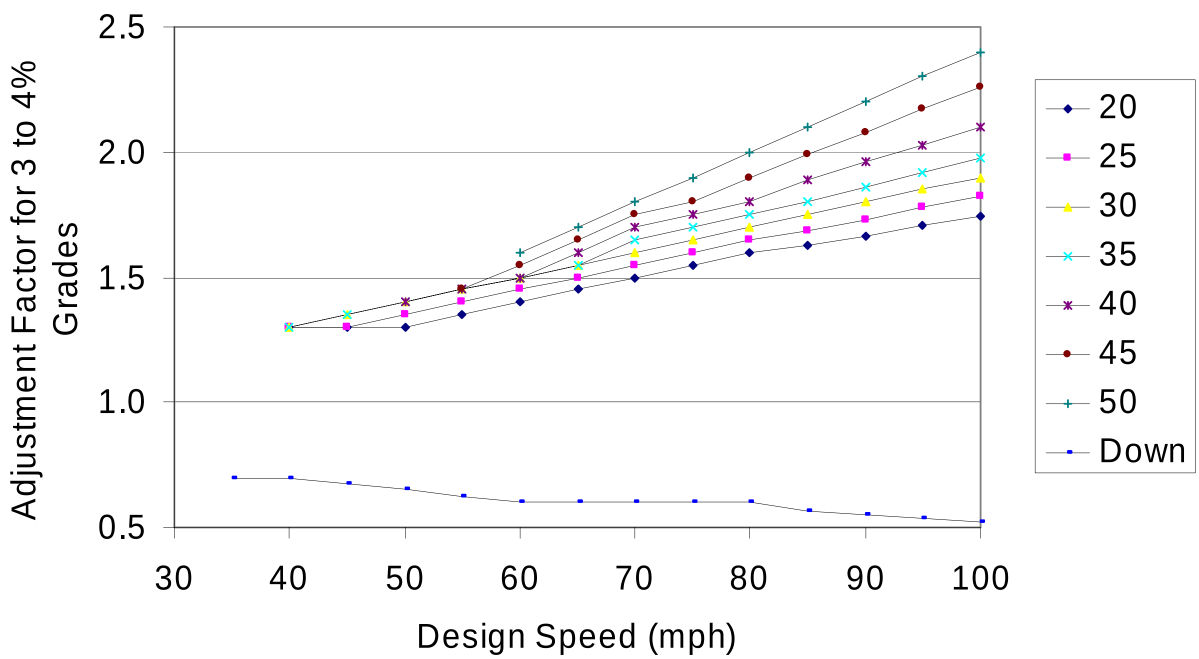
(Metric)						
Design Speed of Roadway (km/h)	Ratio of Length on Grade to Length on Level [a]					
	40	50	60	70	80	All Speeds
	3 to 4% Upgrade					3 to 4% Downgrade
140	1.57	1.67	1.81	1.79	1.90	0.55
150	1.60	1.70	1.86	1.83	1.94	0.53
160	1.63	1.73	1.90	1.86	1.99	0.52
	5 to 6 % Upgrade					5 to 6 % Downgrade
140	2.55	2.82	3.53	3.92	4.42	0.44
150	2.70	3.00	3.81	4.29	4.88	0.42
160	2.86	3.18	4.09	4.65	5.34	0.41

[a] Ratio in this table multiplied by length of acceleration distances gives length of acceleration distance on grade. Shaded areas reflect high-design-speed potential values.



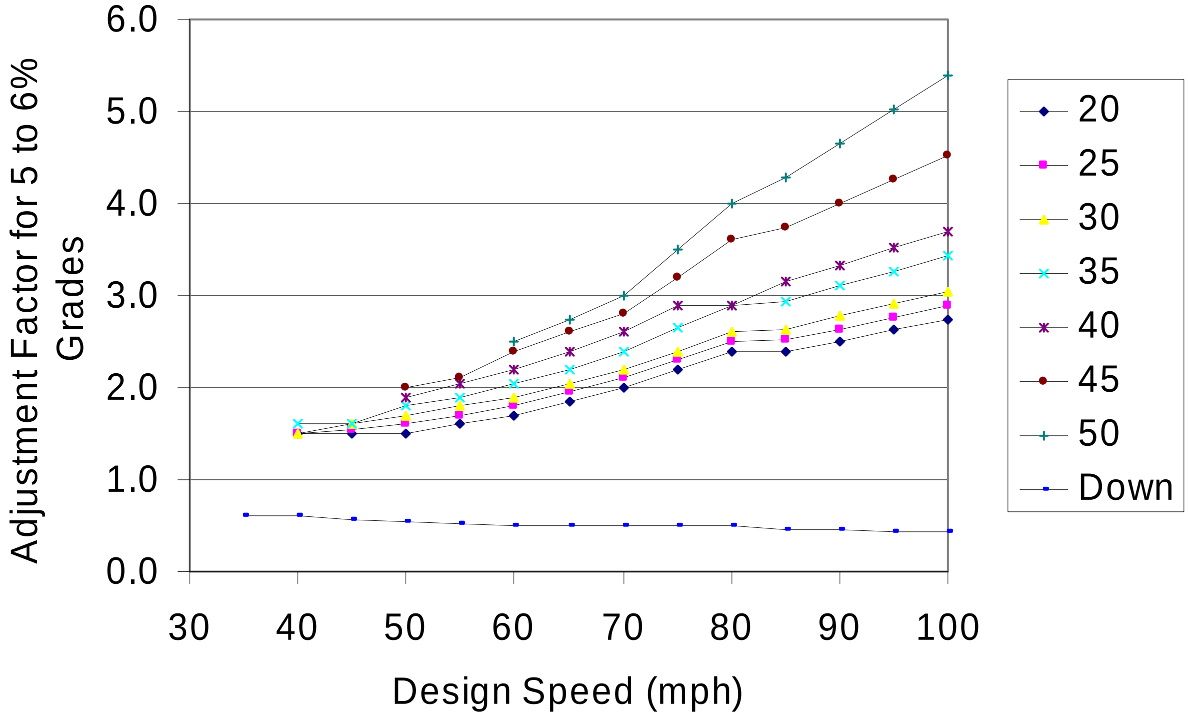
Note: 1 mph = 1.61 km/h

Figure 14-8. Adjustment Factors for Acceleration Lanes on 3 to 4 Percent Grades.



Note: 1 mph = 1.61 km/h

Figure 14-9. Adjustment Factors (Existing and Potential) for Acceleration Lanes on 3 to 4 Percent Grades.



Note: 1 mph = 1.61 km/h

Figure 14-10. Adjustment Factors (Existing and Potential) for Acceleration Lanes on 5 to 6 Percent Grades.

RESEARCH NEEDS

Areas that could benefit from additional research include the following:

- The research team has identified the following assumptions that should be considered, questioned, and perhaps revised with respect to calculating deceleration and acceleration lengths:
  - The speed of the highway vehicle when entering the deceleration lane is less than the design speed of the highway. Currently, the procedures use the "average running speed" from the 1965 *Blue Book* (see Table 14-7 for a reproduction of the *Blue Book* values). So, for a 70-mph [113 km/h] highway design speed, the assumption is that the vehicle enters the deceleration lane traveling at 58 mph [93 km/h] – a 12-mph [20 km/h] difference. Should the design speed, posted speed, or anticipated 85th percentile speed of the freeway be used rather than average running speed?
  - A similar concern exists for acceleration lengths. The acceleration length procedure not only uses average running speed instead of the design speed of the highway, it also assumes that vehicles will be entering the highway at 5 mph [8 km/h] less than the average running speed. So a vehicle entering a 70-mph [113 km/h] freeway is assumed to enter at a speed of only 53 mph [85 km/h] – a 17-mph [28 km/h]

difference. Again, should the design speed, posted speed, or anticipated 85th percentile speed of the freeway be used rather than average running speed?

  - The speed when entering the exit curve is assumed to be less than the design speed of the curve (e.g., the curve may be designed for 40 mph [64 km/h] and the procedure assumes drivers will decelerate to 34 mph [55 km/h] before entering the curve). This conservative assumption does not match observations of typical operations.
  - For 30- and 35-mph [48 and 56 km/h] design speed, the assumed running speed is slightly less for an exit curve as compared to the value assumed for a roadway (see Table 14-7). For 30 mph [48 km/h], the assumed running speed at an intersection is 26 mph [42 km/h]. This is the value that is used for the speed entering the horizontal curve on a ramp within all AASHTO policies. For a 30-mph [48 km/h] highway design speed, an average running speed of 28 mph [45 km/h] was used in the *Blue Book* (a 2-mph [3.2 km/h] difference). While the differences are minor, and the use of 30 mph for a *highway* design speed could be questioned, the differences could create some confusion over which value is appropriate.
  - Deceleration and acceleration rates were estimated from graphs in the 1965 *Blue Book*. Are more current rates available and are they applicable to freeway ramp operations? Do rates vary by initial speed or can a constant speed be used to simplify the procedure?
  - The deceleration procedure assumes drivers will coast for 3 seconds prior to using their brakes. Do exiting drivers coast for less or more time? Does it vary with the type of ramp?
  - At some time between the writing of the 1965 *Blue Book* and the 1973 *Red Book*, the assumption about whether deceleration occurred within the taper section was changed because the taper length dimension was removed from the deceleration and acceleration distances (taper length was included in the deceleration and acceleration lengths in 1965 but not in 1973). Do drivers decelerate in the taper section? Should the assumption be included in the procedure?
  - The TRDM figure (TRDM Figure 3-36), but not the 2004 *Green Book*, states "where providing desirable deceleration length is impractical, it is acceptable to allow for a moderate amount of deceleration (10 mph [16 km/h]) within the through lanes and to consider the taper as part of the deceleration length." If the belief is that the operating speed on the freeway is closer to the design speed than the average running speed (which is supported by research findings), then the procedure assumes a speed reduction is already occurring in the through lanes (i.e., the difference between design speed and average running speed). Because the TRDM does not list the average running speed, users may not appreciate that the procedure already includes the assumption that speed reduction is occurring in the through lane. Of course, the procedure may be so conservative drivers do not need to decelerate in the through lanes to achieve the appropriate exit speed for the downstream curve.
- The basis for the current deceleration and acceleration lengths needs review and updating. An ongoing NCHRP project is tasked with those efforts. Therefore, a new procedure for determining deceleration and acceleration may produce results that will have an impact on the values for high design speeds. Once the new procedure is available, Texas should investigate whether the results are reasonable for the state and how they will impact values being used for high-design-speed roads.

- The TRDM includes taper lengths ranging from 150 to 330 ft [46 to 101 m]. Should these values be retained or should a fixed length be used based on the width of the added (or diminishing) pavement? The TRDM includes a note that a 50:1 or 70:1 rate is to be used when acceleration lengths are greater than 1300 ft [400 m]. Is there an acceptable taper rate for when acceleration lengths are less than 1300 ft [400 m]?
- The adjustment factors used to adjust acceleration and deceleration lengths should be investigated for accuracy. Do the values reflect the capabilities of current vehicles?
- Should the acceleration and deceleration along with adjustment factors be a function of percent trucks?

CHAPTER 15

POTENTIAL DECELERATION AND ACCELERATION RAMP LENGTHS FOR ALL HIGHWAY SPEEDS

The deceleration and acceleration lengths provided in Chapter 14 were developed based on extrapolating the current procedures into the higher design speeds. Each procedure, however, should be investigated to determine if it needs to be replaced with a different model, or on a smaller scale effort, if updates to key assumptions would produce more logical dimensions. A large scale national research project is under way (*52*) to investigate how best to determine acceleration and deceleration lengths. This section presents initial thoughts and findings from a smaller scale effort where key assumptions are updated using information available in the literature.

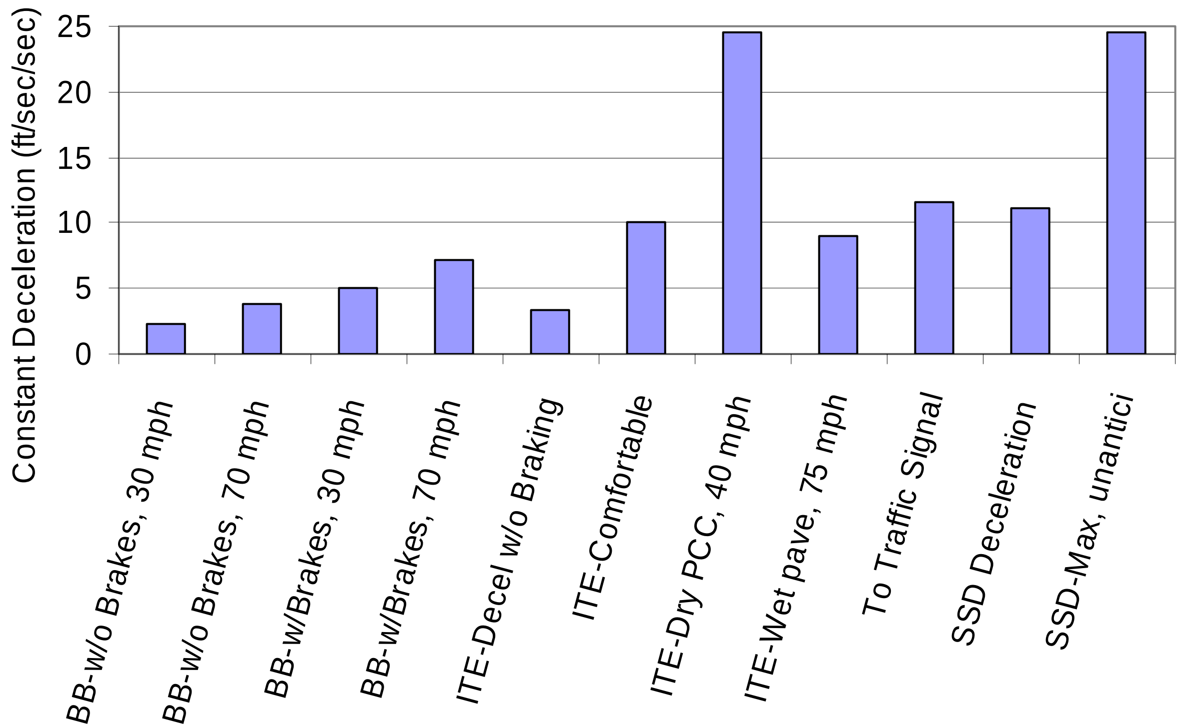
DECELERATION LENGTHS BASE ASSUMPTIONS

The procedure used to generate the deceleration lengths included in the 1965 *Blue Book* (*24*) is based upon: (a) speeds on limited access roads, (b) speeds on ramps, and (c) deceleration behavior. The assumptions within these areas are dated and more current information is needed.

Previous research (*54*, *58*) has demonstrated that speeds on rural two-lane highways routinely have 85th percentile speeds in excess of the inferred design speed of horizontal curves. Other research (*5*, *55*) has demonstrated that 85th percentile operating speeds exceed posted speed on arterial streets and rural highways. Therefore, it would be logical to also assume that drivers exceed the design speed or the posted speed on freeways. Similar concerns exist for the speed at the controlling ramp curve after traversing the deceleration lane. The procedure assumes that drivers will begin the curve at a speed less than the curve's design speed. While conservative, since a longer deceleration length is needed to bring the vehicles speed down to the lower value, it is an assumption that is not currently supported with research findings. The amount of speed reduction, if any, on the freeway prior to entering an auxiliary lane is an area needing research.

While previous research on rural two-lane highways has demonstrated vehicles operate in excess of the design speed of the horizontal and vertical curves, similar speed data for horizontal or vertical curves on freeways are not available. However, a sample of speeds on freeways could indicate if the operating speeds are generally in excess of the assumed running speed. A set of speed data was obtained for four rural Texas freeway sites. The data set included speeds for all vehicles for a one-week period. Focusing on two- and five-axle vehicles, the percent of vehicles in 10-mph [16 km/h] speed bins was determined. Figure 3-2 shows the distribution. Over 55 percent of the two-axle vehicles and about 30 percent of the five-axle vehicles exceeded 70 mph [113 km/h]. These findings, along with the findings from the literature, indicate that operating speed (as measured by 85th percentile speed), or perhaps design or posted speed, is more appropriate than using average running speed in calculating deceleration lengths.

Deceleration behavior on an exit ramp is also an area needing research. The *Blue Book* assumed that drivers will coast for 3 seconds before applying their brakes, and that deceleration is occurring within the taper (24). The *Red Book* modified the assumption that deceleration is occurring within the taper (*53*). However, the *Red Book* maintained the assumption that drivers coast for 3 seconds. The deceleration rates used to create the *Blue Book* values, and the *Green Book* values since the deceleration lengths are similar, were referenced to papers from 1938 and 1940. Figure 15-1 shows a sample of the *Blue Book* (BB) values along with deceleration rates from a sample of other sources.



Note: 1 ft/s^2=0.305 m/s^2

Figure 15-1. Deceleration Rates.

Potential sources for more recent deceleration rates are listed below. A plot of the deceleration rates (all converted to ft/s^2) is shown in Figure 15-1.

- As part of the 1990s study on stopping sight distance (*5*), deceleration during braking maneuvers was recorded. The SSD study measured stopping distances for 26 subjects using an initial speed of 55 mph [89 km/h]. Test conditions included enabled and disabled antilock brakes, wet or dry pavement conditions, and two geometric conditions (tangent section and horizontal curve) for a total of 986 maneuvers. The deceleration rate used in the 2004 *Green Book* stopping sight distance procedure (11.2 ft/s^2 [3.4 m/s^2]) was selected based on the results of that study. A maximum deceleration rate to an unanticipated object was also identified (24.5 ft/s^2 [7.5 m/s^2]) from the SSD study.
- Two studies conducted in the 1980s measured dry-pavement deceleration characteristics to traffic signal change intervals. The study by Chang et al. (*59*) found mean decelerations of 10.5 and 12.5 ft/s^2 [3.2 and 3.8 m/s^2] at the two subject intersections.

Wortman and Matthias *(60)* found mean deceleration at six study sites of 7.0 to 13.0 ft/s^2 [2.1 and 4.0 m/s^2]. The mean value for all observations from the six intersections was 11.6 ft/s^2 [3.5 m/s^2], a result that was consistent with Chang et al.'s findings.

- The ITE *Traffic Engineering Handbook* (*61*) provides a summary of deceleration rates including deceleration without brakes and representative maximum and comfortable decelerations. The deceleration without brakes references the same 1940 study as used to form the basis of the 1965 *Blue Book* values. The representative maximum braking data provided by ITE (see Table 3-12 of the *Traffic Engineering Handbook*) is from a 1948 paper on skid resistance measurements on Virginia highways. It provided data for speeds under 40 mph [64 km/h] for four types of dry surfaces and new and worn tires. The guidance in the ITE *Traffic Engineering Handbook* for the higher speeds was back-calculated from the 1994 *Green Book* stopping sight distance value. The comfortable deceleration advice references a *Green Book* figure that is similar to the figure in the 1965 *Blue Book* (which is based on the 1930s data). A single value of 10 ft/s^2 [3 m/s^2] was also provided as being "reasonably comfortable for occupants of passenger cars" (*61*).

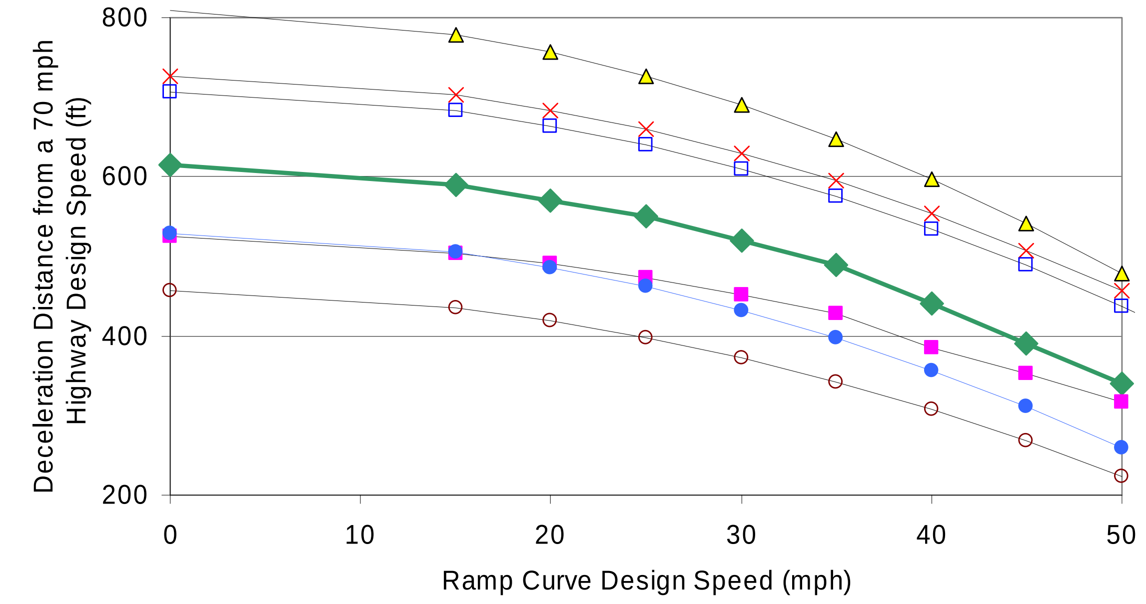
POTENTIAL DECELERATION LENGTHS FOR EXIT TERMINALS

A procedure was identified for calculating the deceleration lengths currently in the 2004 *Green Book* (see Chapter 14). This procedure was used along with modifying the input values for three key assumptions to calculate potential deceleration lengths. The three key assumptions were:

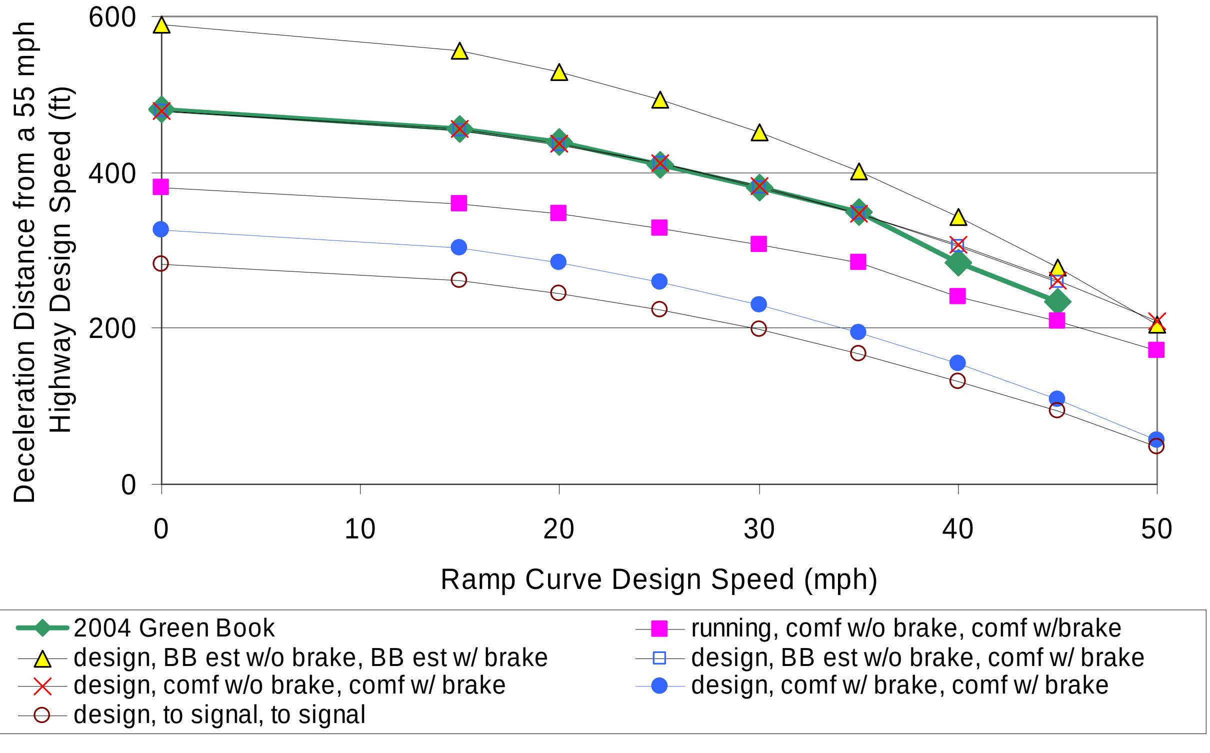
- speeds on the highway and ramp curve,
- initial deceleration, and
- final deceleration.

Deceleration lengths were calculated for several combinations of different assumptions. Figure 15-2 shows the deceleration lengths calculated using different assumptions for speeds and deceleration rates for a 55- and a 70-mph [89 and a 113 km/h] highway design speed to ramp curve design speeds ranging from 0 to 50 mph [0 to 81 km/h]. This figure will assist in illustrating the potential changes in deceleration lengths as a result of changes in the assumptions. The 2004 *Green Book* values are also included so that a comparison can be made. Table 15-1 lists the values used for the different assumptions along with providing an explanation of the abbreviations used in the Figure 15-2 legend.

While more recent deceleration rates are available, their applicability to the exiting maneuver may be questionable. The information is generally presented as only a single value for all initial speeds or it represents only one initial speed (e.g., the deceleration rate for SSD was measured from tests conducted with an initial speed of 55 mph [89 km/h]). The available data from the 1930s may still better represent the exiting maneuver since different deceleration rates can be determined for the difference initial speeds.



70 mph Highway Design Speed



55 mph Highway Design Speed

Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 15-2. Calculated Deceleration Lengths (See Table 15-1 for Explanations of Abbreviations Used in Legend).

Table 15-1. Values Used for the Different Assumptions to Calculate Deceleration Lengths.

Item	Abbreviation Used in Figure 15-2[a]	Value	Source
Running speed	running	See Table 14-7	1965 *Blue Book* (24) 2004 *Green Book* (2)
Design speed	design	Typical range Figure 15-2 uses 55 and 70 mph [89 and 113 km/h]	2004 *Green Book* (2)
Estimated deceleration rate without brakes	BB est. w/o brake	See Table 14-9	Estimated from 1965 *Blue Book* (24) deceleration lengths and deceleration graph
Estimated deceleration rate with brakes	est. w/ brake	See Table 14-9	
Comfortable braking without brakes	Comf w/o brake	3.29 ft/s^2 [1.0 m/s^2]	ITE, page 65 (*61*)
Comfortable braking with brakes	Comf w/ brake	10 ft/s^2 [3.0 m/s^2]	ITE, page 68 (*61*)
Comfortable braking on an approach to a signal change interval	to signal	11.6 ft/s^2 [3.5 m/s^2]	Based on work by Chang et al. (*59*) and Wortman and Matthias (*60*)

[a] The legend in Figure 15-2 shows the three assumptions used to generate the curve: type of speed (running or design), deceleration rate used for initial 3 s deceleration, and deceleration rate used for remaining deceleration.

While appropriate deceleration rates are elusive, the need to update the speed assumption for the highway and the ramp curve is clear. Previous research has demonstrated that drivers are at or exceed the design speed (much less the lower average running speed) on curves. This indicates that the assumption that drivers' speeds are less than design speed in a free-flow situation is certainly questionable; however, there is some evidence that drivers do decelerate in the travel lane before moving into the deceleration lane. A 2006 paper (*62*) reported on a study conducted in Spain that evaluated driver speed and performance changes as a result of reconfigurations of an existing deceleration lane. In the base condition, drivers on the main roadway that were unaffected by the exit maneuver had an 85th percentile speed of 78 mph [125 km/h]. The drivers that were exiting the roadway had an 85th percentile speed of 69 mph [111 km/h] at the point where they had just cleared the mainlane. This represents a 9-mph [14 km/h] difference in speed, or said in another way, represents a deceleration of 9 mph [14 km/h] in the mainlane. The current assumption for a 75-mph [120 km/h] design speed roadway is an average running speed of 61 mph [98 km/h]. Therefore, the running speed assumption should be updated or replaced because even if some deceleration in the mainlane is acceptable, the amount of deceleration appears to be less than the current assumption. If the design procedure assumes no deceleration in the mainlanes, which is a conservative approach, then the speed at the start of the deceleration should be either the design speed or some measure of the operating speed of the facility.

When running speed is replaced by design speed for the highway and for the ramp curve, the deceleration lengths greatly increase, as expected (see triangles as compared to the diamonds in Figure 15-2). Updating the deceleration rate for decelerating with brakes with the more recent finding does decrease the deceleration lengths (see curves with ×s), although they are still greater than or near the current TRDM/2004 *Green Book* values. Updating the braking deceleration rates with more recent information (see open squares in Figure 15-2) results in a decrease in deceleration lengths. Again, the lengths are still greater than the existing values for several speed combinations due to the use of design speed rather than running speed. When the assumption that drivers decelerate without brakes for the initial 3 seconds is replaced with the assumption that drivers decelerate using brakes for the entire length results in deceleration lengths that are less than the values currently included in the TRDM/2004 *Green Book*. The amount of time a driver decelerates without brakes is a key area needing research. The recent study in Spain *(62)* found an average decelerating in gear duration of 6.9 s. Also needed is where the deceleration without braking is occurring – such as in the mainlanes, in the taper, or in the deceleration lane.

Table 15-2 contains the deceleration lengths that would be generated when a 3 second deceleration without braking is used followed by a constant deceleration of 10 ft/s^2 [3 m/s^2] to the ramp design speed. Table 15-3 lists the percent change for each combination of highway design speed and ramp design speed. With only a few exceptions, for highway design speeds of 55 mph [89 km/h] and lower, the deceleration length decreased. Deceleration length increased for design speeds of 60 mph [97 km/h] and greater. Field validation is needed to verify that drivers are using the 10 ft/s^2 [3 m/s^2] deceleration rate along with the 3 seconds of decelerating in gear (i.e., no brake deceleration).

Table 15-2. Potential Deceleration Lengths Using Highway Design and Ramp Design Speeds, *Blue Book* Estimated Deceleration for 3 s followed by a Constant Deceleration (10 ft/s^2 [3 m/s^2]) throughout Remainder of the Exit.

(US Customary)															
Highway Design Speed (mph)	**Deceleration Length (ft) for Exit Curve Design Speed (mph)**														
	Stop	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**	**75**	**80**
30	192	168	149	--	--	--	--	--	--	--	--	--	--	--	--
35	240	216	197	173	--	--	--	--	--	--	--	--	--	--	--
40	293	269	250	226	196	--	--	--	--	--	--	--	--	--	--
45	350	326	307	283	253	218	--	--	--	--	--	--	--	--	--
50	412	388	369	345	315	280	239	--	--	--	--	--	--	--	--
55	478	454	435	411	381	346	306	260	--	--	--	--	--	--	--
60	549	525	506	482	452	417	376	330	279	--	--	--	--	--	--
65	626	602	583	558	529	494	453	407	356	299	--	--	--	--	--
70	707	683	664	640	610	575	535	489	437	318	318	--	--	--	--
75	794	769	750	726	696	661	621	575	523	467	405	337	--	--	--
80	884	860	841	817	787	752	712	666	614	558	496	428	355	--	--
85	980	956	937	912	883	848	807	761	710	653	591	524	451	372	--
90	1080	1056	1037	1013	983	948	907	862	810	754	691	624	551	473	389
95	1181	1156	1137	1113	1083	1048	1008	962	911	854	792	724	651	573	489
100	1280	1256	1237	1213	1183	1148	1108	1062	1010	954	892	824	751	673	589

(Metric)												
Highway Design Speed (km/h)	**Deceleration Length (m) for Exit Curve Design Speed (km/h)**											
	Stop	**20**	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**	**110**	**120**
50	61	56	50	41	--	--	--	--	--	--	--	--
60	80	75	69	60	48	--	--	--	--	--	--	--
70	102	97	90	81	69	55	--	--	--	--	--	--
80	125	120	113	104	92	78	62	--	--	--	--	--
90	151	146	138	129	118	104	87	68	--	--	--	--
100	178	173	166	157	146	131	115	95	73	--	--	--
110	209	204	196	187	175	161	144	125	103	79	--	--
120	241	236	228	219	207	193	176	157	135	111	84	--
130	276	271	262	253	241	227	211	191	169	145	118	88
140	312	307	298	289	278	264	247	228	206	181	154	125
150	350	345	337	328	316	302	285	266	244	220	193	163
160	389	384	377	368	357	343	326	307	285	260	233	204

Table 15-3. Change in Deceleration Lengths Using Values in Table 15-2 Compared to the Values in TRDM/2004 *Green Book*.

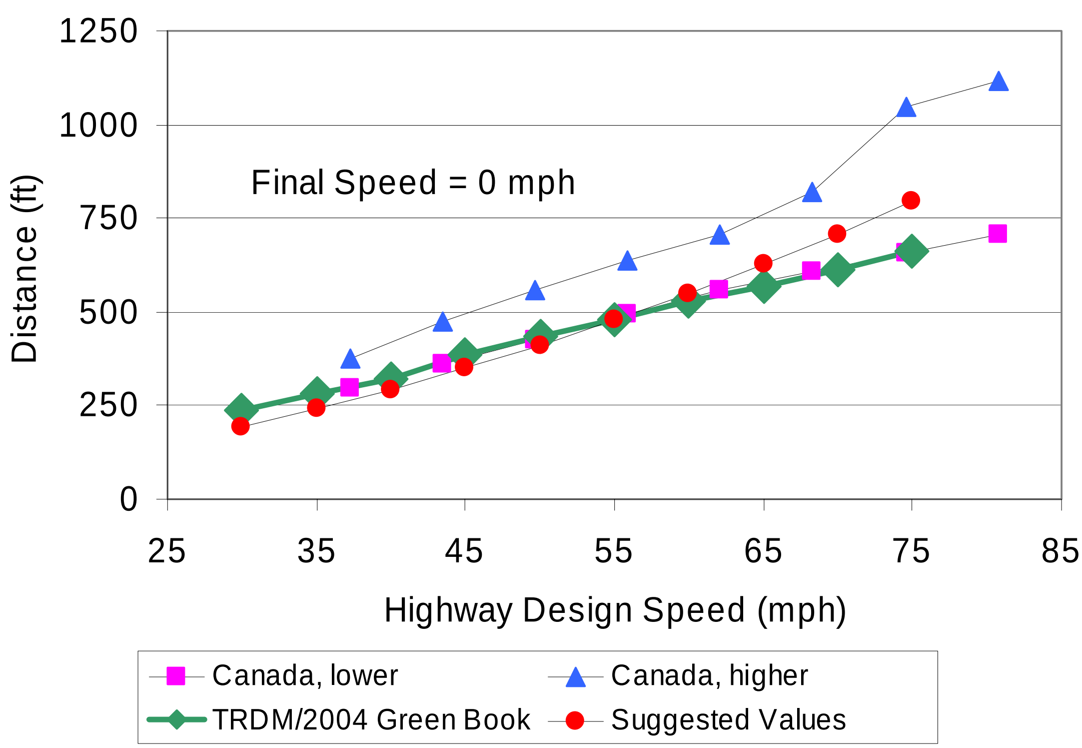
(US Customary)									
Highway Design Speed (mph)	**Change in Deceleration Length (%) for Exit Curve Design Speed (mph)**								
	Stop	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
30	-22	-19	-14	--	--	--	--	--	--
35	-16	-16	-6	-7	--	--	--	--	--
40	-9	-10	-6	-4	6	--	--	--	--
45	-10	-7	-6	-4	1	-1	--	--	--
50	-6	-4	-4	-3	0	-2	6	--	--
55	0	0	-1	0	0	-1	7	9	--
60	3	5	5	4	5	3	7	9	14
65	9	10	11	10	11	11	14	16	21
70	13	14	14	14	15	15	18	20	22
75	17	17	17	17	17	19	21	23	25

(Metric)									
Highway Design Speed (km/h)	**Change in Deceleration Length (%) for Exit Curve Design Speed (km/h)**								
	Stop	**20**	**30**	**40**	**50**	**60**	**70**	**80**	**--**
50	-19	-20	-17	-10	--	--	--	--	--
60	-16	-17	-14	-8	-13	--	--	--	--
70	-8	-8	-6	-5	-1	0	--	--	--
80	-4	-4	-2	4	3	-2	12	--	--
90	3	3	3	8	7	4	16	13	--
100	5	5	7	8	8	9	15	12	--
110	15	12	15	17	17	15	20	19	--
120	20	20	23	25	22	25	26	31	--

The values in Table 2.4.6.2 of the *Geometric Design Guide for Canadian Roads* (63) provide deceleration lengths and samples from that table are reproduced as Table 15-4. Figure 15-3 illustrates the values for decelerating to a stop condition. As shown in Figure 15-3, the recommendations from Table 15-2 are near the lower limit of Canada's for highway design speeds between 30 and 65 mph [48 and 105 km/h]. Above 65-mph [105 km/h] highway speeds, the suggested values are within the Canadian range.

Table 15-4. Reproduction of a Sample of *Geometric Design Guide for Canadian Roads* Table 2.4.6.2 (63).

Speed Design {Assumed Operating} (mph[km/h])	Taper Length (ft[m])	Length of Acceleration Lane Excluding Taper (ft[m]) Design Speed of Turning Roadway Curve (mph[km/h])			
		0 [0]	19 [30]	31 [50]	43 [70]
37 {34-37} [60{55-60}]	180 [55]	295-377 [90-115]	262-344 [80-105]	180-197 [55-60]	
43 {39-43} [70{63-70}]	213 [65]	361-475 [110-145]	328-426 [100-130]	246-344 [75-105]	
50 {43-50} [80{70-80}]	230 [70]	426-557 [130-170]	377-525 [115-160]	311-426 [95-130]	
56 {48-56} [90{77-90}]	262 [80]	492-639 [150-195]	443-590 [135-180]	377-525 [115-160]	262-361 [80-110]
62 {53-62} [100{85-100}]	279 [85]	557-705 [170-215]	508-672 [155-205]	443-607 [135-185]	328-475 [100-145]
68 {57-68} [110{91-110}]	295 [90]	607-820 [185-250]	557-787 [170-240]	492-721 [150-220]	393-623 [120-190]
75 {61-75} [120{98-120}]	311 [95]	656-1049 [200-320]	607-1016 [185-310]	557-984 [170-300]	443-885 [135-270]
81 {65-81} [130{105-130}]	328 [100]	705-1115 [215-340]	656-1082 [200-330]	590-1049 [180-320]	492-934 [150-285]



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 15-3. Comparison of Deceleration Lengths from Canada, TRDM/2004 *Green Book*, and Suggested Values (1, *2*, *63*).

ADJUSTMENT FACTOR FOR EXIT TERMINALS BASE ASSUMPTIONS

The procedure used to generate the adjustment factors included in the 1965 *Blue Book* could not be determined.

POTENTIAL ADJUSTMENT FACTORS FOR EXIT TERMINALS

The 2004 *Green Book* provides equations to calculate stopping sight distances for different grades (2). This methodology was applied to the equations used to calculate deceleration lengths to determine deceleration lengths for different grades. The ratio between the deceleration length on a grade to the deceleration length on level can form the basis for adjustment factors for deceleration. The equations used to determine deceleration lengths on a grade are listed below.

US Customary

$$D = 1.47V_h t_n - 0.5 \times 32.2\left(\frac{d_n}{32.2} + \frac{G}{100}\right)\left(t_n\right)^2 + \frac{\left(1.47V_r\right)^2 - \left(1.47V_a\right)^2}{2 \times 32.2\left(\frac{d_{wb}}{32.2} + \frac{G}{100}\right)} \qquad \textbf{(15-1,US)}$$

$$V_a = \frac{1.47V_h + d_n t_n}{1.47} \qquad \textbf{(15-2,US)}$$

Where:

D = Deceleration lane length, ft;
V_h = Highway speed, mph;
V_a = Speed after t_n seconds of deceleration without brakes, mph;
V_r = Entering speed for controlling exit ramp curve, mph;
t_n = Deceleration time without brakes (assumed to be 3 s), s;
d_n = Deceleration rate without brakes, ft/s^2;
d_{wb} = Deceleration rate with brakes, ft/s^2; and
G = Grade/100.

Metric

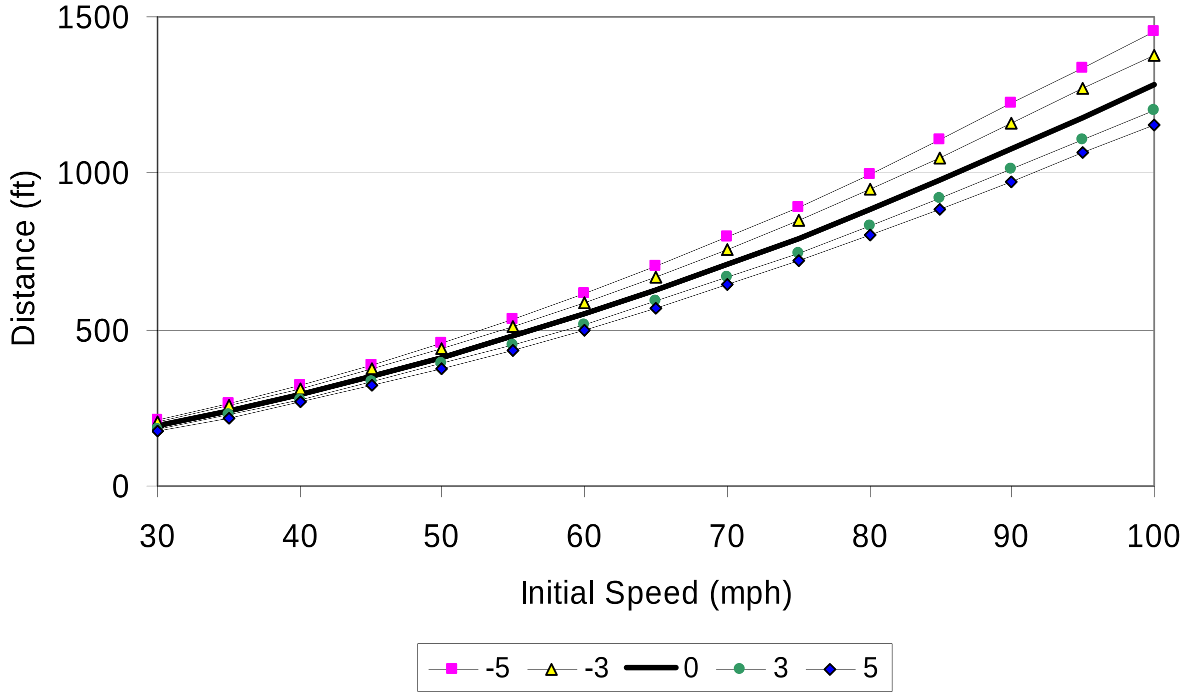
$$D = 0.278V_h t_n - 0.5 \times 9.81\left(\frac{d_n}{9.81} + \frac{G}{100}\right)(t_n)^2 + \frac{(0.278V_r)^2 - (0.278V_a)^2}{2 \times 9.81\left(\frac{d_{wb}}{9.81} + \frac{G}{100}\right)} \qquad \textbf{(15-1, Metric)}$$

$$V_a = \frac{0.278V_h + d_n t_n}{0.278} \qquad \textbf{(15-2, Metric)}$$

Where:

D = Deceleration lane length, m;
V_h = Highway speed, km/h;
V_a = Speed after t_n seconds of deceleration without brakes, km/h;
V_r = Entering speed for controlling exit ramp curve, km/h;
t_n = Deceleration time without brakes (assumed to be 3 s), s;
d_n = Deceleration rate without brakes, m/s^2;
d_{wb} = Deceleration rate with brakes, m/s^2; and
G = Grade/100.

Figure 15-4 shows the distances traveled for a passenger car decelerating from an initial speed to a stop on different grades (3, 5, 0, -3, and -5 percent). The percent change in distance was fairly consistent across all initial speeds. For example, for the -5 percent grade, the percent increase ranged from 11 percent increase at a 30-mph [48 km/h] initial speed to a 13 percent increase in distance for a 100-mph [161 km/h] initial speed. The small differences in percent change make using only one adjustment factor per grade for all initial speeds logical. It also simplifies the use of adjustment factors for deceleration lanes (as compared to acceleration lanes) and mirrors the current approach to deceleration adjustment factors as contained in the TRDM/2004 *Green Book*. While the approach is similar to what is in the TRDM/2004 *Green Book*, the magnitude of the adjustment factor values is quite different. Figure 15-5 illustrates the potential adjustment factors derived using the above equations. The adjustment factor for each grade was determined as the average of the calculated adjustment factors for initial speeds ranging from 30 mph [48 km/h] to 100 mph [161 km/h]. An exception was made for grades of 2 and -2 percent. These grades were assumed to have no influence on the decelerating distances, even though some effect can be seen in Figure 15-5. Table 15-5 lists the potential adjustment factors.



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 15-4. Deceleration Distance for a Passenger Car Traveling from an Initial Speed to a Stop on Different Percent Grades.

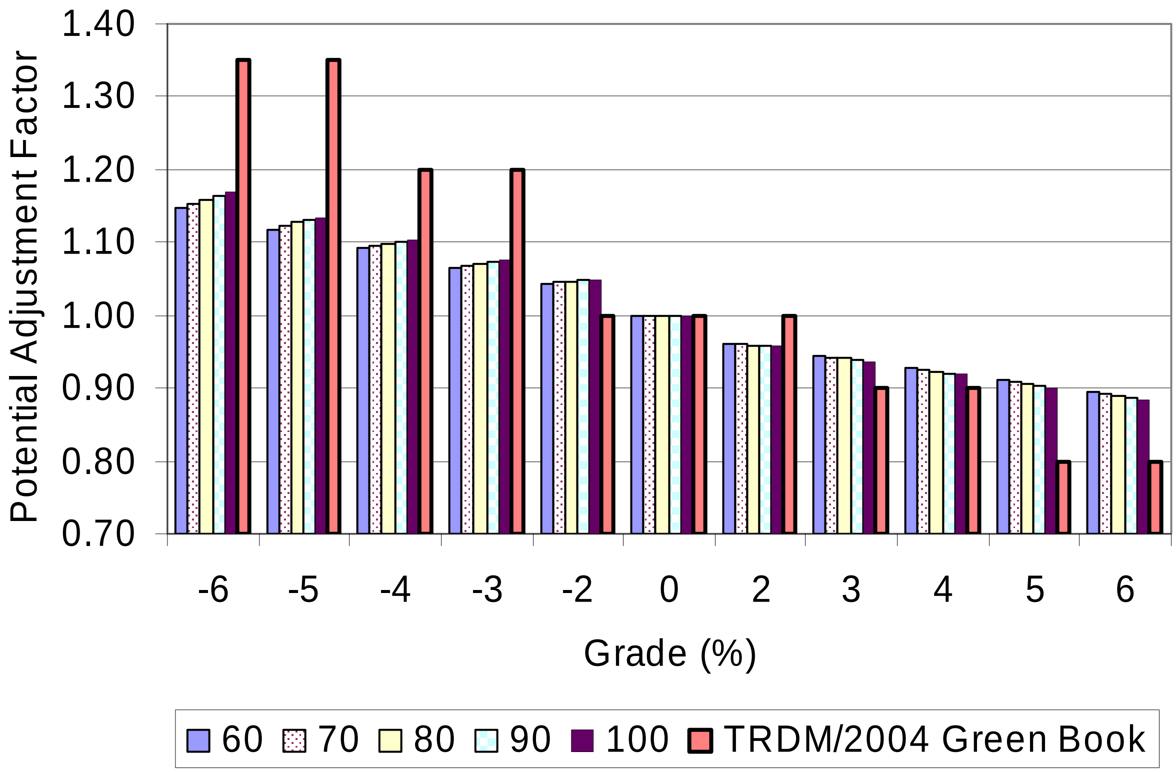


Figure 15-5. Comparison of Potential Adjustment Factors for Different Initial Speeds and Grades.

Table 15-5. Potential Adjustment Factors for Deceleration Lanes.

Design Speed of Roadway	Ratio of Length on Grade to Length on Level								
	-6%	-5%	-4%	-3%	-2 to 2%	3%	4%	5%	6%
ALL	1.15	1.12	1.09	1.07	1.00	0.94	0.93	0.91	0.89

TAPER LENGTH BASE ASSUMPTIONS

Taper length is determined as 3.5 s multiplied by the average running speed. It is based on a 1941 study (*57*).

POTENTIAL TAPER LENGTHS FOR ENTRANCES AND EXITS

If average running speed is being replaced by design speed or operating speed for deceleration or acceleration lengths, then the speed used to calculate taper lengths should also change. Table 15-6 lists the results when design speed is used.

Table 15-6. Potential Taper Lengths Using Design Speed.

(US Customary)			
Design Speed (mph)	**TRDM Taper Lengths (ft) or Suggested Taper Lengths (See Chapter 14)**	**Length of Taper (ft)**	
		Calculated [a]	**Rounded**
30	150	154	155
35	165	180	180
40	180	206	210
45	200	232	235
50	230	257	260
55	250	283	285
60	265	309	310
65	285	334	335
70	300	360	360
75	330	386	390
75	315	See above	See above
80	330	412	415
85	345	437	440
90	360	463	465
95	370	489	490
100	425	515	515

(Metric)			
Design Speed (km/h)	**TRDM Taper Lengths (m) or Suggested Taper Lengths (See Chapter 14)**	**Length of Taper (m)**	
		Calculated [b]	**Rounded**
50	45	49	50
60	55	58	60
70	60	68	70
80	70	78	80
90	75	88	90
100	80	97	100
110	90	107	110
120	100	117	120
120	95	See above	See above
130	100	126	130
140	110	136	140
150	115	146	150
160	130	156	160

[a] Determined using following equation: Taper Length = 3.5 × 1.47 × (design speed in ft/s).
[b] Determined using following equation: Taper Length = 3.5 × 0.278 × (design speed in m/s).
[c] Repeated because a change in value was suggested (see Chapter 14).
Shaded areas reflect high-design-speed potential values.

ACCELERATION LENGTH BASE ASSUMPTIONS

The procedure used to generate the acceleration lengths included in the 1965 *Blue Book* is based upon: (a) speeds on limited-access roads, (b) speeds on ramp, and (c) acceleration behavior. The assumptions within these areas are dated and more current information is needed.

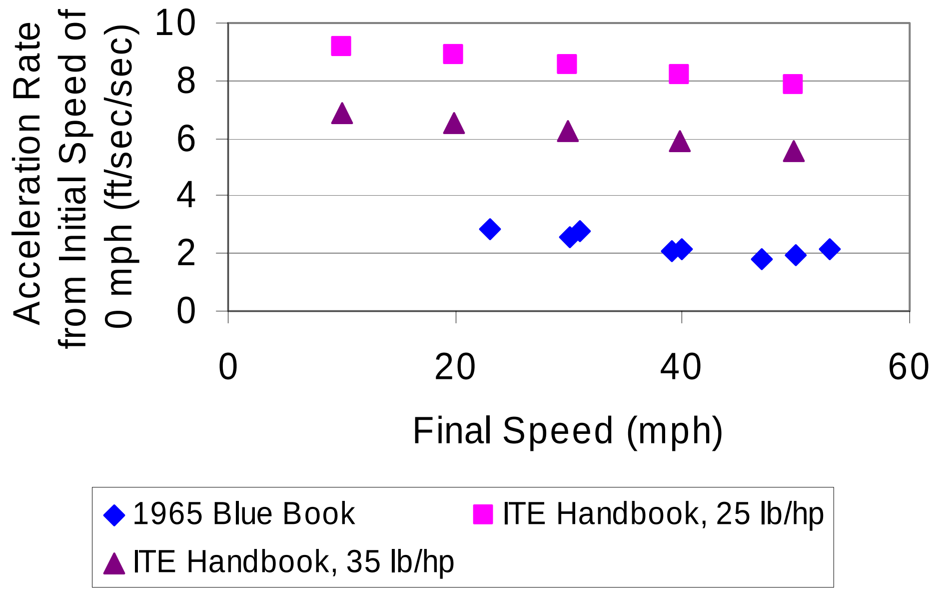
Previous research *(54, 58)* has demonstrated that speeds on rural two-lane highways routinely have 85th percentile speeds in excess of the inferred design speed of horizontal curves. Other research (*5*, *55*) has demonstrated that 85th percentile operating speeds exceed posted speeds on arterial streets and rural highways. Therefore, it would be logical to also assume that drivers may exceed the design speed or the posted speed on limited-access roads. A recent study of 23 entrance ramps in Canada *(64)* found a typical 85th percentile merging speed of 65 mph [105 km/h] for vehicles entering facilities with a 62-mph [100 km/h] posted speed limit.

In summary, assuming that drivers will enter the limited-access road at a speed much less than the free-flow speed is questionable. Similar concerns exist for the speed at the controlling curve on the entrance ramp. The procedure assumes that drivers will exit the curve at a speed less than the curve's design speed. This conservative assumption is not currently supported with research findings or operational experience.

In addition to the speeds on an entrance ramp, acceleration behavior at an entrance ramp is also an area needing research. The acceleration rates used to create the *Blue Book* values, and the *Green Book* values because the acceleration lengths are similar, were referenced to papers from 1938 and 1940. More recent research has identified acceleration rates for both passenger cars *(65)* and trucks *(66)*. A summary of the findings for maximum acceleration is included in the ITE *Traffic Engineering Handbook* Tables 3-9 and 3-10 *(61)*. Some of these findings are compared to the 1965 *Blue Book* values in Figure 15-6. The ITE *Handbook* notes that maximum acceleration rates are seldom used in normal driving and provides a copy of the *Green Book* speed versus distance acceleration figure as a potential source for acceleration rates when "drivers were not influenced to accelerate rapidly" such as passenger cars starting up after a traffic signal turns green and vehicles passing on four-lane highways.

Another potential source for determining distance traveled while accelerating is to use vehicle performance equations. NCHRP Report 505 *(13)* includes discussion on truck characteristics with respect to critical length of grade. The authors developed a spreadsheet as part of their evaluation that can generate a truck speed profile on grade. The spreadsheet was included with their report and could be used to determine distance traveled from an assumed ramp curve speed to a highway speed. There are potential limitations with using a spreadsheet that was developed for trucks without calibrating the equations for passenger cars. However, the use of the procedures may provide an appreciation of the potential changes in the lengths for speed changes if vehicle performance (rather than driver's gap acceptance or other driver characteristics) is the key element.

Vehicle performance *(13, 14, 15, 16)* was reviewed as part of the grade analysis (see Chapter 3). An internal spreadsheet was generated that would easily permit comparisons between different assumptions. This spreadsheet – called the “5544 spreadsheet” – was a tool used to determine potential acceleration lengths for passenger cars as presented below.



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 15-6. Examples of Acceleration Rates from *Blue Book* and ITE *Handbook* (*24*, *61*).

POTENTIAL ACCELERATION LENGTHS FOR ENTRANCE TERMINALS

A procedure was identified for calculating the acceleration lengths currently in the 2004 *Green Book* (see Chapter 14). This procedure was used along with modifying the input values for key assumptions to calculate potential acceleration lengths. The key assumptions were:

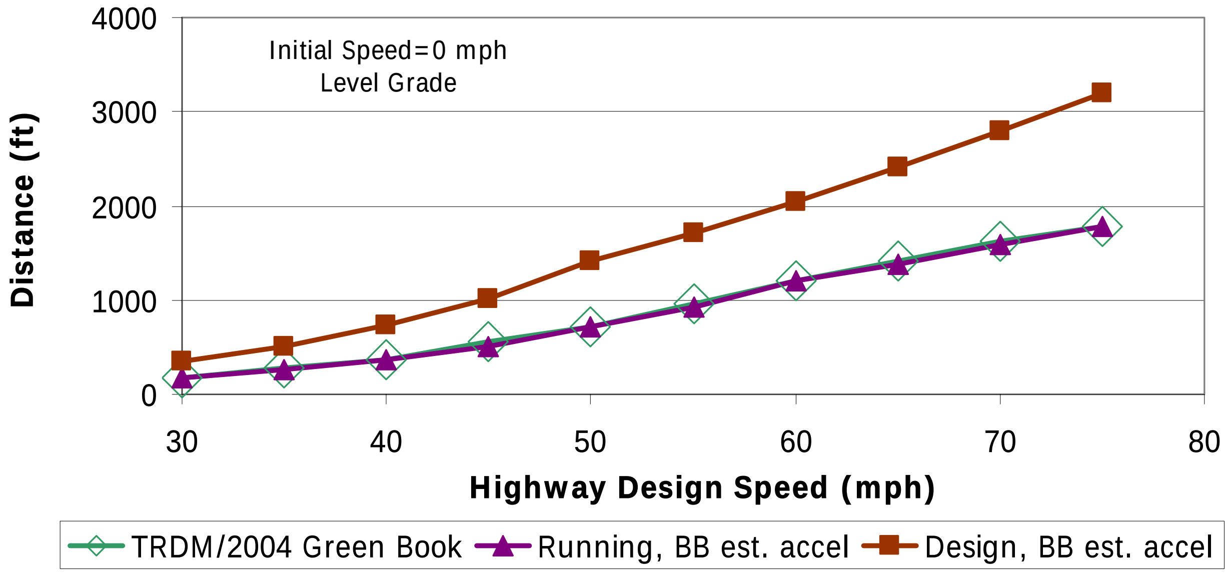
- speeds on the highway and ramp curve and
- acceleration.

Speeds on the highway and ramps were assumed to equal either:

- design speed of the highway and ramp or
- running speed for the given design speed of the highway and the ramp.

As expected, when using design speed rather than running speed within the methodology identified from the *Blue Book*, the acceleration lengths increase greatly (see Figure 15-7). For a highway design speed of 70 mph [113 km/h], the acceleration lengths would change from 1600 to 2800 ft [488 to 854 m]. Using acceleration performance more representative of current vehicles may offset some of the increase caused by using design speed rather than running speed.

Key characteristics of the passenger car/light truck (PC/LT) fleet with respect to acceleration performance have been evolving. Table 15-7 lists the weight and wt/hp values for the new passenger car and light truck fleet for the most recent 20 years available as identified by the National Highway Traffic Safety Administration (NHTSA) (*67*). The curb weight of the vehicle fleet has increased from about 2800 to 3200 lb [1271 to 1453 kg]. The weight to horsepower (HP) ratio has decreased from 26.0 to 18.1 [16 to 11 kg/kw].



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 15-7. Acceleration Lengths When Using Design Speed or Running Speed and the *Blue Book* Procedure.

Two spreadsheets are available that provide second-to-second acceleration for a vehicle: the 5544 spreadsheet and the NCHRP 505 spreadsheet. Each spreadsheet has different input requirements. The following criteria were selected for use in the spreadsheets:

- 5544 spreadsheet:
  - weight of 2004 PC/LT = 3200 lb [1453 kg],
  - weight of 1986 PC/LT = 2800 lb [1271 kg],
  - weight/power ratio of 2004 PC/LT = 18.1 lb/hp [11.0 kg/kw],
  - weight/power ratio of 1986 PC/LT = 25.7 lb/hp [15.6 kg/kw],
  - erag coefficient = 0.35,
  - rrontal area of vehicle = 40 ft^2 [3.7 m^2], and
  - mixed tires coefficients.

- NCHRP 505 spreadsheet:
  - weight/power ratio = 18.1 lb/hp [11.0 kg/kw].

Table 15-7. NHTSA Passenger Car and Light Truck Fleets Characteristics (67).

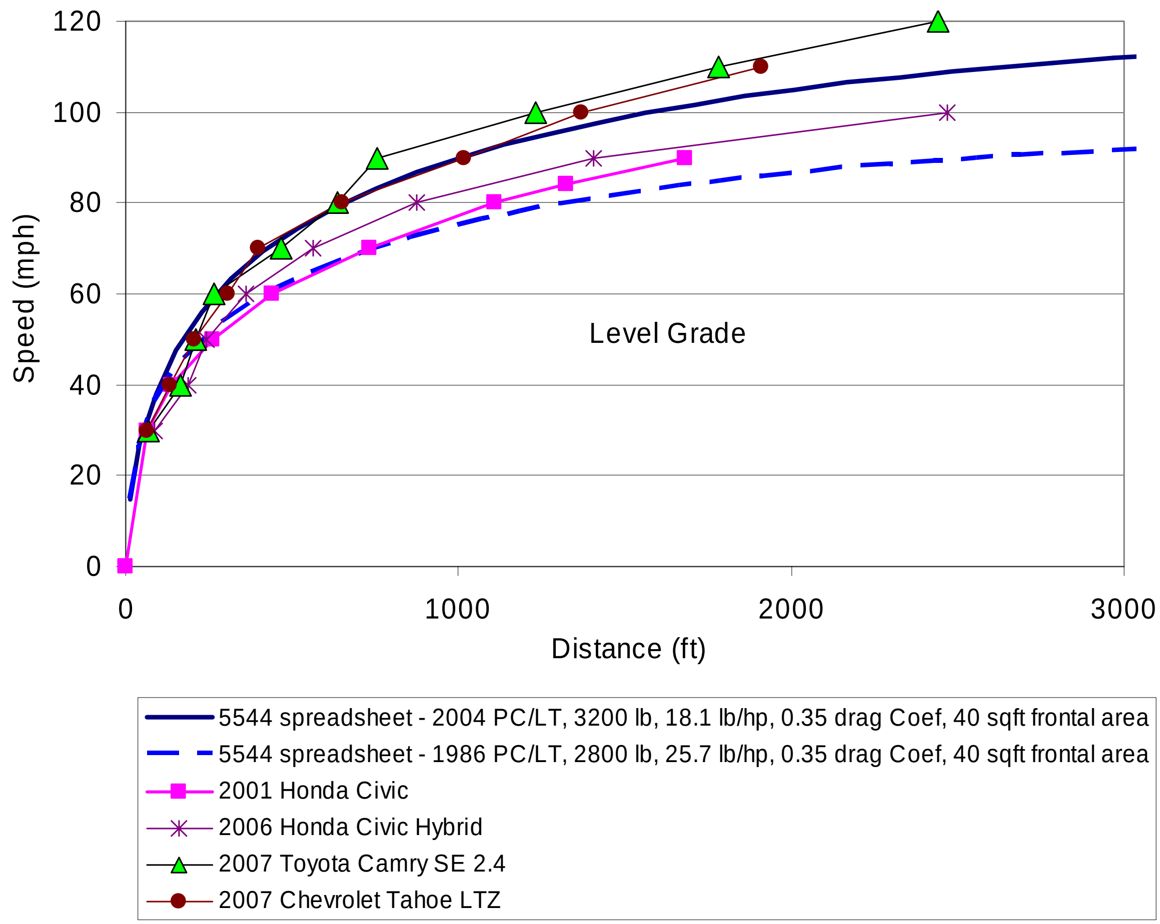
Year	CAFÉ[a] (mpg) [kmpg]	Curb Weight (lb) [kg]	HP/Curb Weight [kw/kg]	Weight/HP[b] [kg/kw]
1985	27.6 [44.4]	2867 [1302]	0.0384 [0.0632]	26.0 [15.8]
1986	28.2 [45.4]	2821 [1281]	0.0389 [0.0640]	25.7 [15.7]
1987	28.5 [45.9]	2805 [1273]	0.0398 [0.0655]	25.1 [15.3]
1988	28.8 [46.4]	2831 [1285]	0.0411 [0.0677]	24.3 [14.8]
1989	28.4 [45.7]	2879 [1307]	0.0428 [0.0703]	23.4 [14.2]
1990	28.0 [45.1]	2906 [1319]	0.0453 [0.0744]	22.1 [13.4]
1991	28.4 [45.7]	2934 [1332]	0.0442 [0.0727]	22.6 [13.7]
1992	27.9 [44.9]	3007 [1365]	0.0456 [0.0751]	21.9 [13.3]
1993	28.4 [45.7]	2971 [1349]	0.0462 [0.0761]	21.6 [13.1]
1994	28.3 [45.6]	3011 [1367]	0.0479 [0.0787]	20.9 [12.7]
1995	28.6 [46.0]	3047 [1383]	0.0487 [0.0802]	20.5 [12.5]
1996	28.5 [45.9]	3049 [1384]	0.0493 [0.0810]	20.3 [12.3]
1997	28.7 [46.2]	3071 [1394]	0.0495 [0.0814]	20.2 [12.3]
1998	28.8 [46.4]	3075 [1396]	0.0505 [0.0830]	19.8 [12.0]
1999	28.3 [45.6]	3116 [1415]	0.0521 [0.0856]	19.2 [11.7]
2000	28.5 [45.9]	3126 [3148]	0.0525 [0.0865]	19.0 [11.6]
2001	28.8 [46.4]	3148 [1429]	0.0530 [0.0870]	18.9 [11.5]
2002	29.0 [46.7]	3163 [1436]	0.0539 [0.0884]	18.6 [11.3]
2003	29.5 [47.5]	3179 [1443]	0.0548 [0.0903]	18.2 [11.1]
2004	29.1 [46.9]	3239 [1471]	0.0554 [0.0908]	18.1 [11.0]

[a] CAFE = Corporate average fuel economy.
[b] Determined as 1/(HP/Curb Weight).

While PC/LT characteristics were selected for use in the 5544 spreadsheet, there were still concerns on whether the spreadsheet would reasonably predict vehicle performance because comfortable acceleration to high speeds is not readily available. *Car and Driver* magazine includes time and distance data for selected vehicles, as well as other performance characteristics and vehicle dimensions. The data for a 2001 Honda Civic (68) along with a sample of more recent vehicles (69, 70, 71, 72) were compared to the findings from the 5544 spreadsheet as shown in Figure 15-8. Generally, recent vehicle models reached 90 mph [145 km/h] in less than 1000 ft [305 m], except for a hybrid vehicle that required 1400 ft [427 m]. The average 1986 PC/LT vehicle would require a greater distance to reach 90 mph [145 km/h] and the 2004 PC/LT vehicle would require less distance. The improvements in wt/hp resulted in the shorter acceleration distances. It also resulted in the 2004 vehicle surpassing 100 mph [161 km/h] while the top speed of the 1986 vehicle was 94 mph [151 km/h].

Data for high-performance vehicles were also identified from the Internet (73). The time to accelerate to 100 mph [161 km/h] from a stop ranged between 7.4 seconds (2005 Ford GT) to 11.1 seconds (1995 Toyota Supra). Converting the 11.1 seconds into distance traveled resulted in the 1995 Toyota Supra reaching 100 mph [161 km/h] in 816 ft [249 m]. The 5544 spreadsheet predicts that about 2100 ft [641 m] is needed for a 2004 PC/LT to accelerate to 100 mph [161 km/h].

The results from the 5544 spreadsheet in general have a pattern that matches the acceleration pattern for the test track results. While the pattern appears reasonable, the distances calculated for the 2004 PC/LT fleet may represent optimal conditions rather than the acceleration distances that drivers would use while merging onto a limited access facility. Therefore, the results from the vehicle performance equations should be verified with in-field observations.

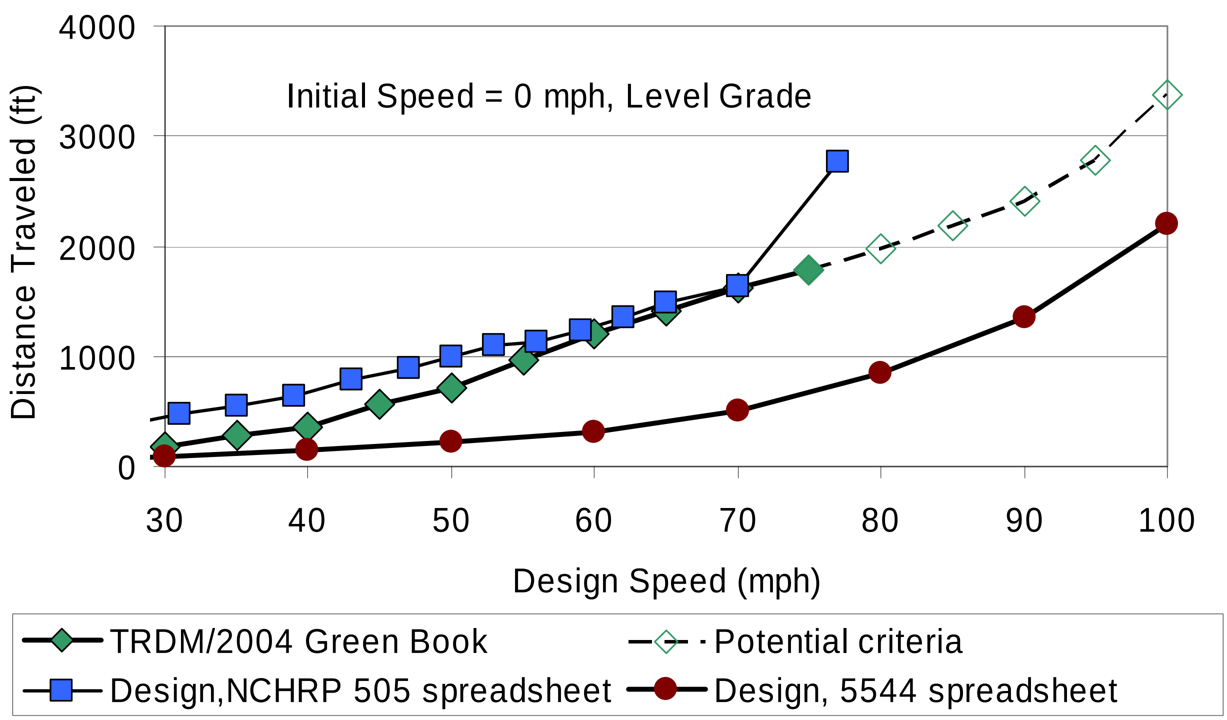


Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 15-8. Comparison of 5544 Technique Results for a 1986 and 2004 Passenger Car with Time and Distance Data.

Figure 15-9 compares the results from the two spreadsheets using 2004 PC/LT characteristics to the TRDM/2004 *Green Book* values. The vehicle performance equations used in the 5544 spreadsheet provided acceleration lengths that were much less than the current TRDM/2004 *Green Book* values and the proposed extrapolated values (see Chapter 14). For 70-mph [113 km/h] design speed, the 5544 spreadsheet would result in an acceleration length of only 530 ft [162 m] versus 1620 ft [494 m] for the TRDM/2004 *Green Book*. Again, caution should be expressed that the results from the 5544 spreadsheet may be generating optimal acceleration distances rather than a reflection of driver's needs when merging onto a limited access facility.

The NCHRP Report 505 provides a spreadsheet that can be used to develop a speed profile for trucks. The equations were developed using truck performance, such as accounting for gear shift delays when a truck is coasting with no power supplied by the engine. Using an 18.1-lb/hp [11.0 kg/kw] ratio, rather than a typical truck value, such as 200 lb/hp [122 kg/kw], resulted in acceleration lengths that exceeded the values in the existing TRDM/2004 *Green Book* and the extrapolated values (see Figure 15-9). Even when 100 mph [161 km/h] was entered as the desired speed, the top speed predicted by the NCHRP 505 spreadsheet for the passenger car was 79 mph [127 km/h]. Because the NCHRP Report 505 equations were developed for trucks, they should have assumptions that must be modified to accurately predict passenger car speeds. The equations used within the 5544 spreadsheet did include consideration of passenger car characteristics. These results require validation, however.



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 15-9. Acceleration Lengths When Using Design Speed and Results from Vehicle Performance Spreadsheets.

While using equations to predict the second-to-second acceleration performance for a vehicle provides a better reflection of actual operations, using a constant acceleration rate may provide a reasonable approximation of the needed acceleration length with significantly less effort. Constant acceleration rates are available from several sources. The following sources were used in a comparison:

- The ITE *Traffic Engineering Handbook* (61) provides examples of normal and maximum acceleration rates for passenger cars and trucks. The maximum acceleration for passenger cars from a stop position is based on a 1978 NCHRP report (65). The figure for the normal acceleration rates was a reference to the speed versus distance plot included in the AASHTO *Green Book*. The rate selected from the ITE *Handbook* for comparison with other acceleration assumptions is the maximum acceleration rate for a 35-lb/hp [21 kg/kw] passenger car on a level road going from 0 to 30, 40, or 50 mph [0 to 48, 64, or 81 km/h] (acceleration rates of 6.23, 5.91, and 5.58 ft/s^2 [1.9, 1.8, 1.7 m/s2], respectively).

- The maximum acceleration rates from the ITE *Handbook* (61) were extrapolated to higher design speeds. The extrapolated rates were from 5.26 ft/s^2 [1.60 m/s^2] at 60 mph [97 km/h] to 3.96 ft/s^2 [1.21 m/s^2] at 100 mph [161 km/h].
- Constant acceleration rates of 3 ft/s^2 [0.91 m/s^2] and 2.35 ft/s^2 [0.72 m/s^2] were selected based on a TxDOT study that examined ramp design (56). The acceleration rates were calculated from spot speeds determined from time/distance measurements pulled from a video of the entrance ramp. Table 15-8 lists the observed ramp driver acceleration rates found for five ramps – three of which were classified as having "poor" geometrics and two with "good" geometrics. The larger acceleration rates associated with ramps with poor geometrics were theorized as being the result of drivers feeling forced to use unusual trajectories in order to negotiate problematic limited-access road entry facilities. The value of 3 ft/s^2 [0.91 m/s^2] was selected as a rounded average mean 85th percentile acceleration rate for all ramps observed in the study. The 2.35 ft/s^2 [0.72 m/s^2] represent the mean 85th percentile acceleration rate for ramps with good geometrics. While a preference would be to use the average, or perhaps even the 15th percentile value (i.e., use an acceleration rate that 85 percent or more of the drivers would accept), these numbers were not available within the report. The acceleration rates probably do not include the initial portion of the ramp (e.g., speeds from a stop position to 30 mph [48 km/h] or 50 mph [81 km/h]) based upon a review of plots included in the appendices. Therefore, the reported 85th percentile rates may be different than what the values would have been if the initial acceleration had been included.

Table 15-8. Observed Ramp Driver Acceleration Rates (56).

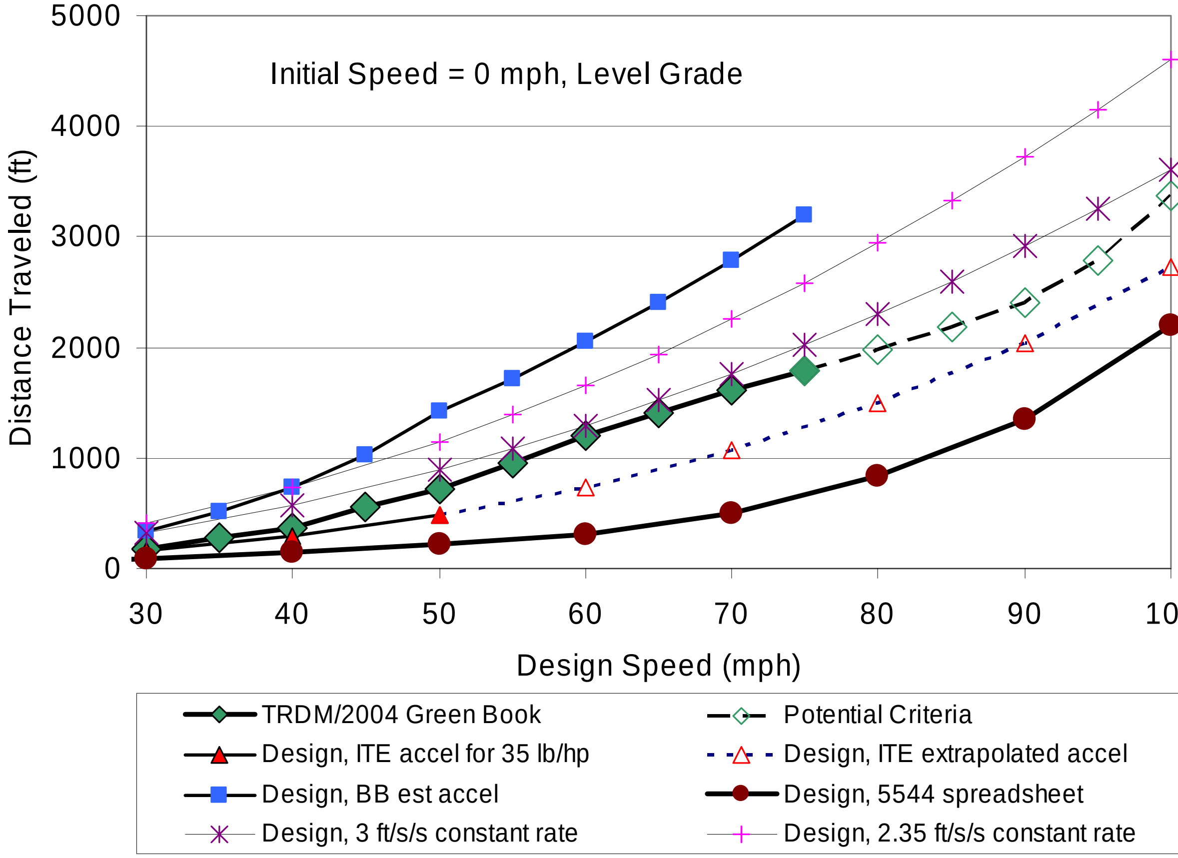
Geometric Category [a]	Ramp	85th Percentile Acceleration Rate (ft/s^2 [m/s^2])		
		Grand Mean	**Maximum**	**Minimum**
Poor	Site A	3.08 [0.94]	7.63 [2.33]	-2.79 [-0.85]
	Site B	3.96 [1.21]	7.19 [2.19]	0.73 [0.22]
	Site C	3.96 [1.21]	8.95 [2.73]	0.59 [0.18]
	Mean	**3.67 [1.12]**	**7.92** [2.41]	**-0.44** [-0.13]
Good	Site D	2.64 [0.80]	6.01 [1.83]	0.15 [0.05]
	Site E	2.05 [0.62]	3.37 [1.03]	1.17 [0.36]
	Mean	**2.35 [0.72]**	**4.69** [1.43]	**0.73** [0.22]

[a] Ramps were categorized as "good" if the ramp design generally exceeded all AASHTO design criteria and "poor" if the ramp failed to meet current criteria.

Figure 15-10 shows the resulting acceleration lengths when using the constant rates as described in the bulleted list above. The acceleration values currently in the TRDM/2004 *Green Book* along with the values predicted using the 5544 spreadsheet and the values predicted using the *Blue Book* acceleration rates are also shown for comparison. The values assumed for the constant accelerations resulted in acceleration lengths that were near or above the current values with the exception of the distances calculated using the ITE maximum acceleration rates. For a 70-mph [113 km/h] design speed the acceleration length would change from 1620 ft [494 m] to 2250 ft [686 m] using an assumed 2.35 ft/s^2 [0.72 m/s^2] rate. Using a 3 ft/s^2 [0.91 m/s^2] rate, the acceleration length would change from 1620 ft [494 m] to 1765 ft [538 m]. Additional in-field observations or greater in-depth investigation is needed to determine if a constant acceleration assumption is reasonable at the higher speeds.

A recent Canadian study *(64)* collected speed and merging behavior at 23 entrance ramps. Continuous speeds were measured using laser speed guns during off-peak periods. The vehicles were generally tracked from the ramp gore to the merging point on a highway with a 62-mph [100 km/h] posted speed. The following conclusions were made from the study:

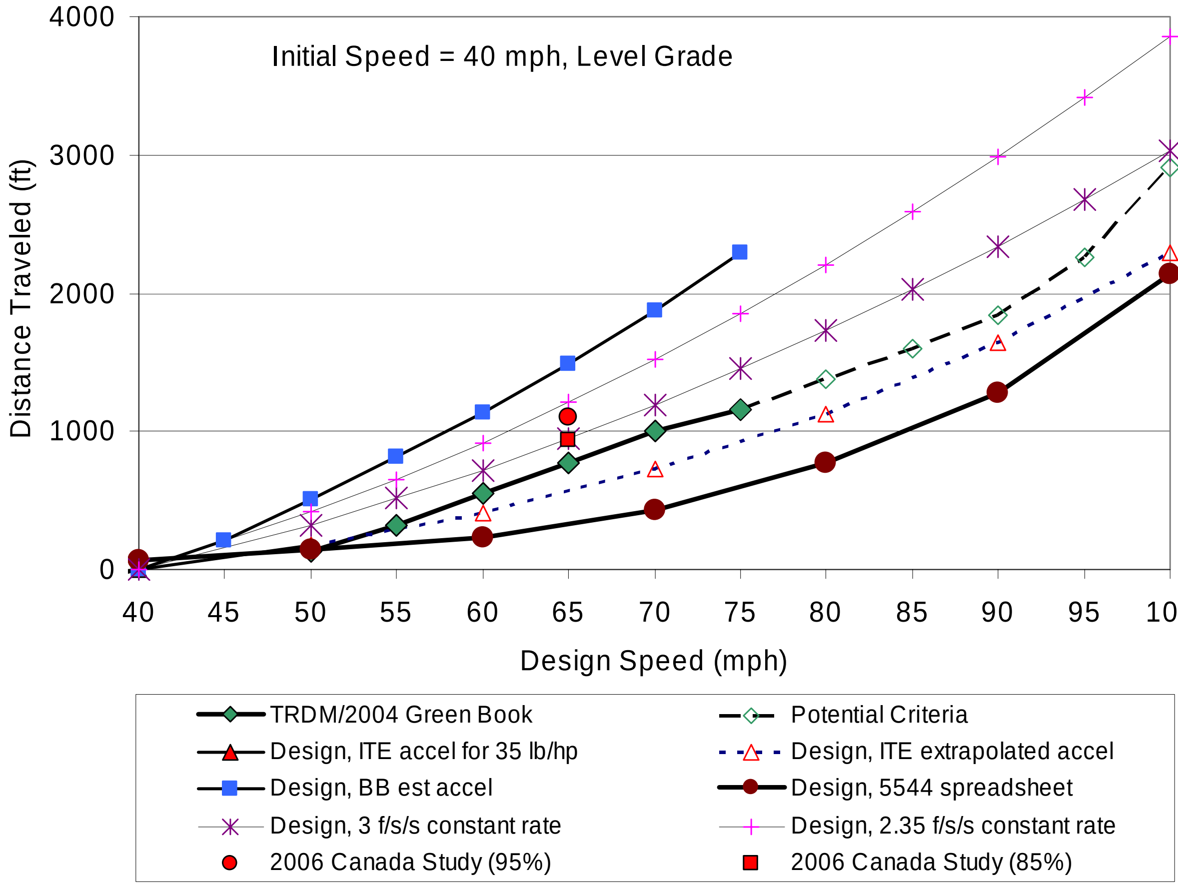
- The 85th percentile maximum comfortable acceleration rate was observed to be 6.6 ft/s^2 [2.0 m/s^2]. The authors identified, upon request, the 15th percentile acceleration rate. The acceleration rate ranged between 2.6 and 5.0 ft/s^2 [0.78 and 1.62 m/s^2] with an average rate of 3.5 ft/s^2 [1.07 m/s^2].
- The 85th percentile merging speed was about 65 mph [105 km/h] regardless of the existing acceleration length. The typical merging speed ranged between 57 and 71 mph [91 and 115 km/h] for the 23 entrance ramps.
- A minimum speed change lane length of 1230 ft [375 m] from the point when the ramp and mainline pavement edges are 4.1 ft [1.25 m] apart to the end of taper is desired for comfortable merging maneuvers of 85 percent of the entering vehicles. The 1230 ft [375 m] includes 300 ft [90 m] for a taper, so the acceleration length would be 935 ft [285 m].
- To accommodate 95 percent of entering vehicles, the acceleration length would be 1100 ft [336 m].
- The lowest 85th percentile gore speed was 42 mph [67.9 km/h]. Speed measurements began near the gore so speeds upstream of that point were not reported.



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 15-10. Acceleration Lengths from a Stop Using Constant Acceleration in Comparison to 2004 *Green Book* and 5544 Spreadsheet Results.

The findings from the Canadian study were compared to existing values in the TRDM/2004 *Green Book*. From the study, the starting speed was 42 mph [67.6 km/h] and the ending speed was 65 mph [105 km/h]. Rounding the 42 mph [67 km/h] to 40 mph [64 km/h] and assuming the speeds are representative of the design speeds of the facility would result in an acceleration length of 770 ft [235 m] from the *Green Book*. The Canadian findings suggest that longer acceleration lengths are needed than what are currently in the *Green Book*. Figure 15-11 illustrates the findings using the constant acceleration values as discussed above along with the findings from the Canadian study. The recommended distance of 935 ft [285 m] accommodates 85 percent of the drivers and represents a 3.0-ft/s^2 [0.9 m/s^2] acceleration rate. The recommended distance of 1100 ft [336 m] accommodates 95 percent of the drivers and represents a 2.5-ft/s^2 [0.8 m/s^2] acceleration rate.



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

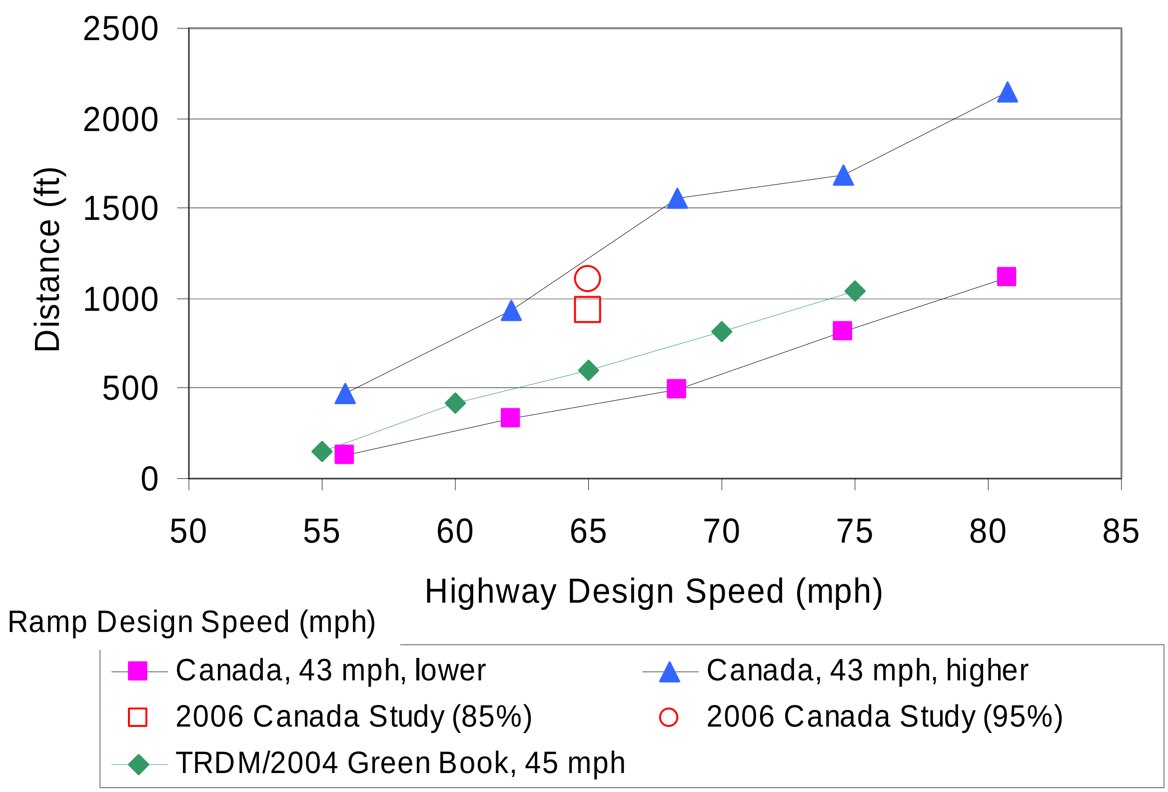
Figure 15-11. Acceleration Lengths from 40 mph [64 km/h] Using Constant Acceleration in Comparison to 2004 *Green Book* and 5544 Spreadsheet Results.

While the recommendations from the Canadian study result in greater distances when compared to the TRDM/2004 *Green Book* values, the recommendations are within the range suggested in Table 2.4.6.5 of the *Geometric Design Guide for Canadian Roads* (63) – a portion is reproduced in this chapter as Table 15-9. Figure 15-12 illustrates the values for a 43-mph [70 km/h] ramp design speed. As shown in Figure 15-12, the recommendations from the Canada study are near the upper limit of their criteria, with the finding that accommodates 95 percent of drivers being just at the higher criteria curve.

Table 15-9. Reproduction of a Sample of *Geometric Design Guide for Canadian Roads* Table 2.4.6.5 (*63*).

Speed Design {Assumed Operating} (mph[km/h])	TL[a]	Length of Acceleration Lane Excluding Taper (ft[m])			
		Design Speed of Turning Roadway Curve (mph[km/h])			
		0 [0]	19 [30]	31 [50]	43 [70]
37 {34-37} [60{55-60}]	180 [55]	279-377 [85-115]	197-262 [60-80]	66-115 [20-35]	
43 {39-43} [70{63-70}]	213 [65]	393-525 [120-160]	328-443 [100-135]	164-279 [50-85]	
50 {43-50} [80{70-80}]	230 [70]	525-738 [160-225]	426-656 [130-200]	279-525 [85-160]	
56 {48-56} [90{77-90}]	262 [80]	705-1066 [215-325]	590-984 [180-300]	459-820 [140-250]	131-475 [40-145]
62 {53-62} [100{85-100}]	279 [85]	902-1475 [275-450]	787-1377 [240-420]	656-1230 [200-375]	328-934 [100-285]
68 {57-68} [110{91-110}]	295 [90]	1082-2131 [330-650]	1000-2066 [305-630]	852-1885 [260-575]	492-1557 [150-475]
75 {61-75} [120{98-120}]	311 [95]	1344-2393 [410-730]	1230-2328 [375-710]	1115-2164 [340-660]	820-1689 [250-515]
81 {65-81} [130{105-130}]	328 [100]	1803-2902 [550-885]	1672-2852 [510-870]	1541-2689 [470-820]	1115-2148 [340-655]

[a] TL = Taper Length (ft[m])



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 15-12. Comparison of Acceleration Lengths for 45 mph [72 km/h] Ramp Curve Design Speed from TRDM/2004 *Green Book,* 43 mph [69 km/h] Ramp Curve Design Speed from 1999 Canadian *Guide*, and Results from 2006 Canada Study (*1, 2, 63, 64*).

A constant acceleration rate of 2.5 ft/s^2 [0.8 m/s^2] and 3.0 ft/s^2 [0.9 m/s^2] was used along with the highway and curve design speed to generate potential acceleration lengths. The values for an acceleration rate of 2.5 ft/s^2 [0.8 m/s^2] are listed in Table 15-10, and listed in Table 15-11 is the percent increase in acceleration length as compared to the TRDM/2004 *Green Book*. Figure 15-13 plots a sample of the values. Table 15-12 lists the lengths when the acceleration is 3.0 ft/s^2 [0.9 m/s^2] and Table 15-13 lists the percent change as compared to the TRDM/2004 *Green Book* values.

Table 15-10. Potential Acceleration Lengths Using Highway Design and Ramp Design Speeds and Constant Acceleration (2.5 ft/s^2 [0.8 m/s^2]) throughout Entire Entrance.

(US Customary)														
HDS [a] (mph)	**Acceleration Length for Entrance Curve Design Speed (mph)**													
	Stop	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**	**75**
30	389	292	216	--	--	--	--	--	--	--	--	--	--	--
35	529	432	357	259	--	--	--	--	--	--	--	--	--	--
40	691	594	519	421	303	--	--	--	--	--	--	--	--	--
45	875	778	702	605	486	346	--	--	--	--	--	--	--	--
50	1080	983	908	810	691	551	389	--	--	--	--	--	--	--
55	1307	1210	1134	1037	918	778	616	432	--	--	--	--	--	--
60	1556	1459	1383	1286	1167	1026	864	681	475	--	--	--	--	--
65	1826	1729	1653	1556	1437	1297	1134	951	746	519	--	--	--	--
70	2118	2020	1945	1848	1729	1588	1426	1243	1037	810	562	--	--	--
75	2431	2334	2258	2161	2042	1902	1740	1556	1351	1124	875	605	--	--
80	2766	2669	2593	2496	2377	2237	2074	1891	1686	1459	1210	940	648	--
85	3123	3025	2950	2852	2734	2593	2431	2247	2042	1815	1567	1297	1005	691
90	3501	3403	3328	3231	3112	2971	2809	2625	2420	2193	1945	1675	1383	1070
95	3900	3803	3728	3630	3511	3371	3209	3025	2820	2593	2345	2074	1783	1469
100	4322	4225	4149	4052	3933	3792	3630	3447	3241	3014	2766	2496	2204	1891

(Metric)												
HDS [a] (km/h)	**Acceleration Length for Entrance Curve Design Speed (km/h)**											
	Stop	**20**	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**	**110**	**120**
50	121	101	77	--	--	--	--	--	--	--	--	--
60	174	155	130	97	--	--	--	--	--	--	--	--
70	237	217	193	159	116	--	--	--	--	--	--	--
80	309	290	266	232	188	135	--	--	--	--	--	--
90	391	372	348	314	270	217	155	--	--	--	--	--
100	483	464	440	406	362	309	246	174	--	--	--	--
110	584	565	541	507	464	411	348	275	193	--	--	--
120	686	676	652	618	575	522	459	386	304	213	--	--
130	816	797	773	739	696	543	580	507	425	333	232	--
140	947	927	903	869	826	773	710	638	555	464	362	251
150	1087	1067	1043	1010	966	913	850	778	696	604	502	391
160	1237	1217	1193	1159	1116	1063	1000	927	845	754	652	541

[a] HDS = Highway Design Speed

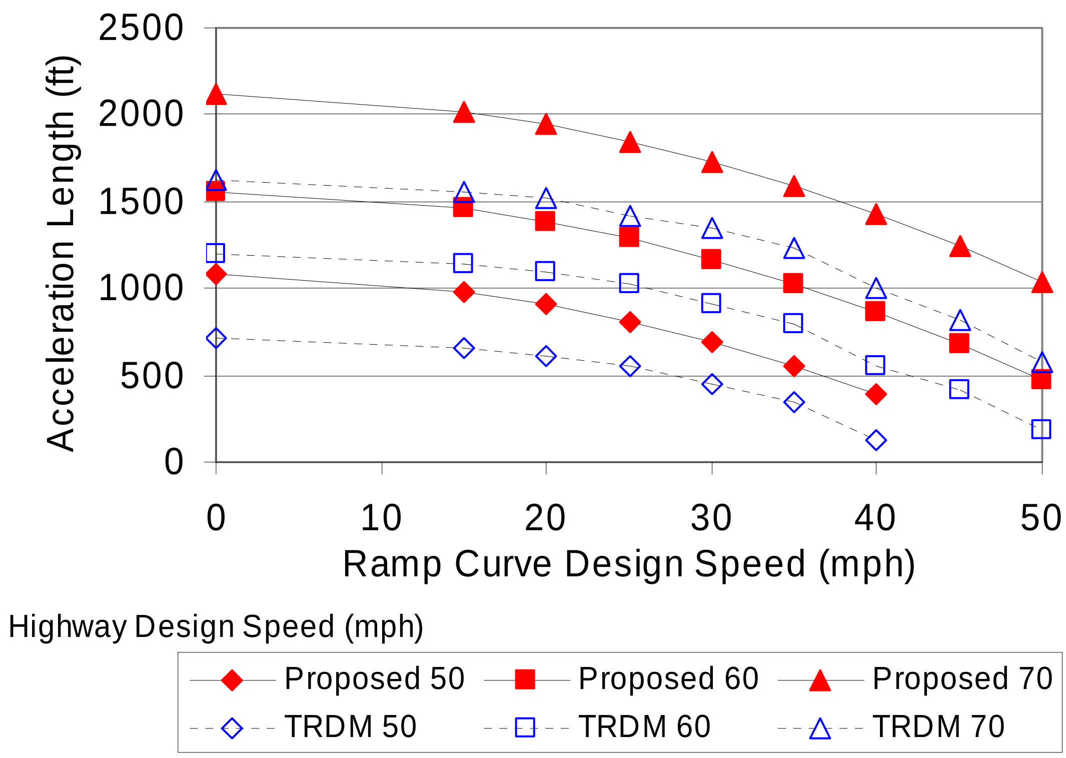
Table 15-11. Change in Acceleration Lengths Using Highway Design and Ramp Design Speeds and Constant Acceleration (2.5 ft/s^2 [0.8 m/s^2]) throughout Entire Entrance as Compared to the Values in TRDM/2004 Green Book.

(US Customary)

Highway Design Speed (mph)	Percent (%) Increase in Acceleration Length for Entrance Curve Design Speed (mph)								
	Stop	15	20	25	30	35	40	45	50
30	116	108	--	--	--	--	--	--	--
35	89	96	123	--	--	--	--	--	--
40	92	98	92	101	152	--	--	--	--
45	56	59	60	59	74	116	--	--	--
50	50	49	49	47	54	57	199	--	--
55	36	34	40	33	37	41	92	188	--
60	30	28	26	26	28	28	57	62	164
65	30	28	26	28	28	39	47	58	101
70	31	30	28	30	28	28	43	52	79
75	36	35	39	37	35	34	50	50	73

(Metric)

Highway Design Speed (km/h)	Percent (%) Increase in Acceleration Length for Entrance Curve Design Speed (km/h)							
	Stop	20	30	40	50	60	70	80
50	101	103	158	--	--	--	--	--
60	83	93	101	115	--	--	--	--
70	58	67	76	77	78	--	--	--
80	55	61	61	60	64	108	--	--
90	50	52	55	53	55	74	342	--
100	40	43	44	42	42	51	124	335
110	36	38	39	37	36	42	74	120
120	28	28	27	26	25	27	41	58



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 15-13. Plot of Potential Acceleration Lengths Using Constant Acceleration of 2.5 ft/s^2 [0.8 m/s^2] Compared to TRDM/2004 *Green Book*.

Table 15-12. Potential Acceleration Lengths Using Highway Design and Ramp Design Speeds and Constant Acceleration (3.0 ft/s^2 [0.9 m/s^2]) throughout Entire Entrance.

(US Customary)

Highway Design Speed (mph)	Change in Acceleration Length (%) for Entrance Curve Design Speed (mph)													
	Stop	15	20	25	30	35	40	45	50	55	60	65	70	75
30	324	243	180	--	--	--	--	--	--	--	--	--	--	--
35	441	360	297	216	--	--	--	--	--	--	--	--	--	--
40	576	495	432	351	252	--	--	--	--	--	--	--	--	--
45	729	648	585	504	405	288	--	--	--	--	--	--	--	--
50	900	819	756	675	576	459	324	--	--	--	--	--	--	--
55	1089	1008	945	864	765	648	513	360	--	--	--	--	--	--
60	1297	1216	1152	1071	972	855	720	567	396	--	--	--	--	--
65	1522	1441	1378	1297	1197	1080	945	792	621	432	--	--	--	--
70	1765	1684	1621	1540	1441	1324	1188	1035	864	675	468	--	--	--
75	2026	1945	1882	1801	1702	1585	1450	1297	1125	936	729	504	--	--
80	2305	2224	2161	2080	1981	1864	1729	1576	1405	1216	1008	783	540	--
85	2602	2521	2458	2377	2278	2161	2026	1873	1702	1513	1306	1080	837	576
90	2917	2836	2773	2692	2593	2476	2341	2188	2017	1828	1621	1396	1152	891
95	3250	3169	3106	3025	2926	2809	2674	2521	2350	2161	1954	1729	1486	1225
100	3602	3520	3457	3376	3277	3160	3025	2872	2701	2512	2305	2080	1837	1576

(Metric)

Highway Design Speed (km/h)	Acceleration Length for Entrance Curve Design Speed (km/h)											
	Stop	20	30	40	50	60	70	80	90	100	110	120
50	107	90	69	--	--	--	--	--	--	--	--	--
60	155	137	116	86	--	--	--	--	--	--	--	--
70	210	193	172	142	103	--	--	--	--	--	--	--
80	275	258	236	206	167	120	--	--	--	--	--	--
90	348	331	309	279	240	193	137	--	--	--	--	--
100	429	412	391	361	322	275	219	155	--	--	--	--
110	520	502	481	451	412	365	309	245	172	--	--	--
120	618	601	580	550	511	464	408	343	270	189	--	--
130	726	708	687	657	618	571	515	451	378	296	206	--
140	842	824	803	773	734	687	631	567	494	412	322	223
150	966	949	927	897	859	811	756	691	618	537	447	348
160	1099	1082	1061	1030	992	945	889	824	751	670	580	481

Table 15-13. Change in Acceleration Lengths Using Highway Design and Ramp Design Speeds and Constant Acceleration (3.0 ft/s^2) throughout Entire Entrance as Compared to the Values in TRDM/2004 Green Book.

Highway Design Speed (mph)	Percent (%) Increase in Acceleration Length for Entrance Curve Design Speed (mph)								
	Stop	15	20	25	30	35	40	45	50
30	80	74	--	--	--	--	--	--	--
35	58	64	86	--	--	--	--	--	--
40	60	65	60	67	110	--	--	--	--
45	30	32	33	33	45	80	--	--	--
50	25	24	24	23	28	31	149	--	--
55	13	12	17	11	14	18	60	140	--
60	8	7	5	5	7	7	31	35	120
65	8	7	5	6	7	8	23	32	68
70	9	8	7	8	7	8	19	26	49
75	13	12	15	14	13	12	25	25	44

(Metric)

Highway Design Speed (km/h)	Percent (%) Increase in Acceleration Length for Entrance Curve Design Speed (km/h)							
	Stop	20	30	40	50	60	70	80
50	79	80	129	--	--	--	--	--
60	63	72	78	91	--	--	--	--
70	40	49	56	57	59	--	--	--
80	37	43	43	472	46	85	--	--
90	34	35	37	36	37	55	293	--
100	24	27	28	27	26	34	99	286
110	21	23	23	22	21	26	55	96
120	13	13	13	12	11	13	26	40

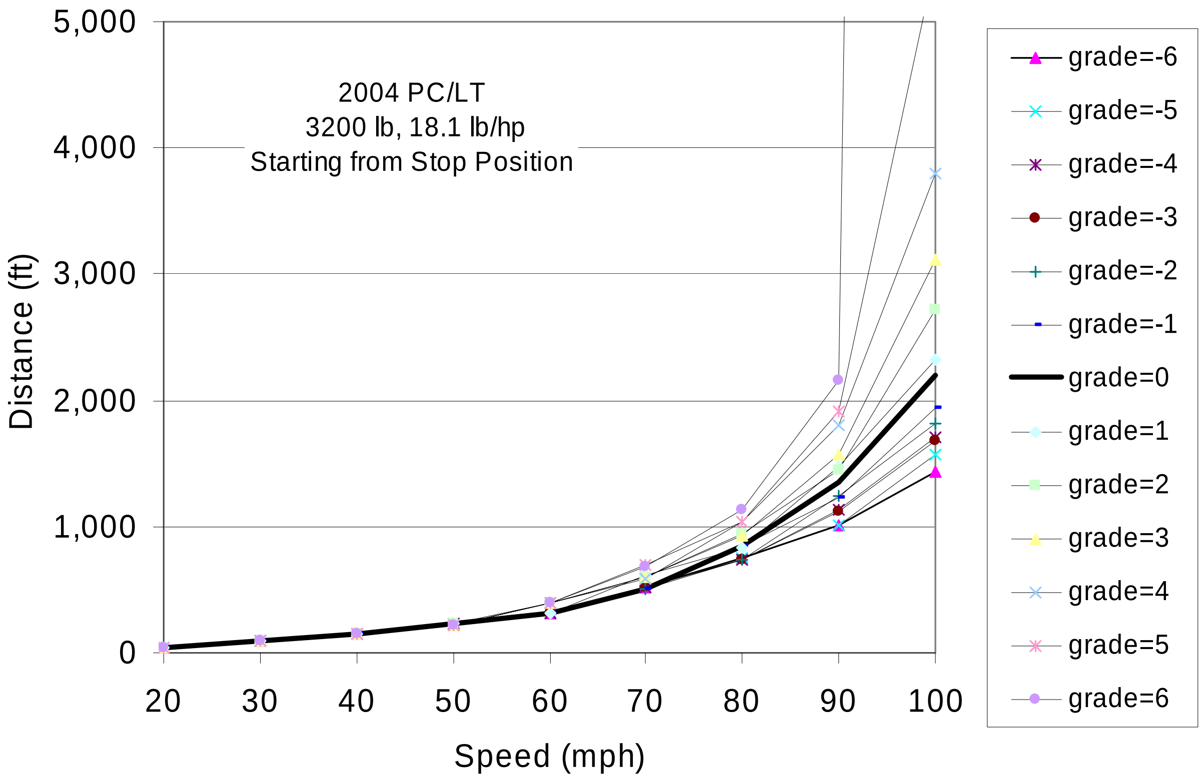
ADJUSTMENT FACTOR FOR ENTRANCE TERMINALS BASE ASSUMPTIONS

The procedure used to generate the adjustment factors included in the 1965 *Blue Book* could not be determined.

POTENTIAL ADJUSTMENT FACTORS FOR ENTRANCE TERMINALS

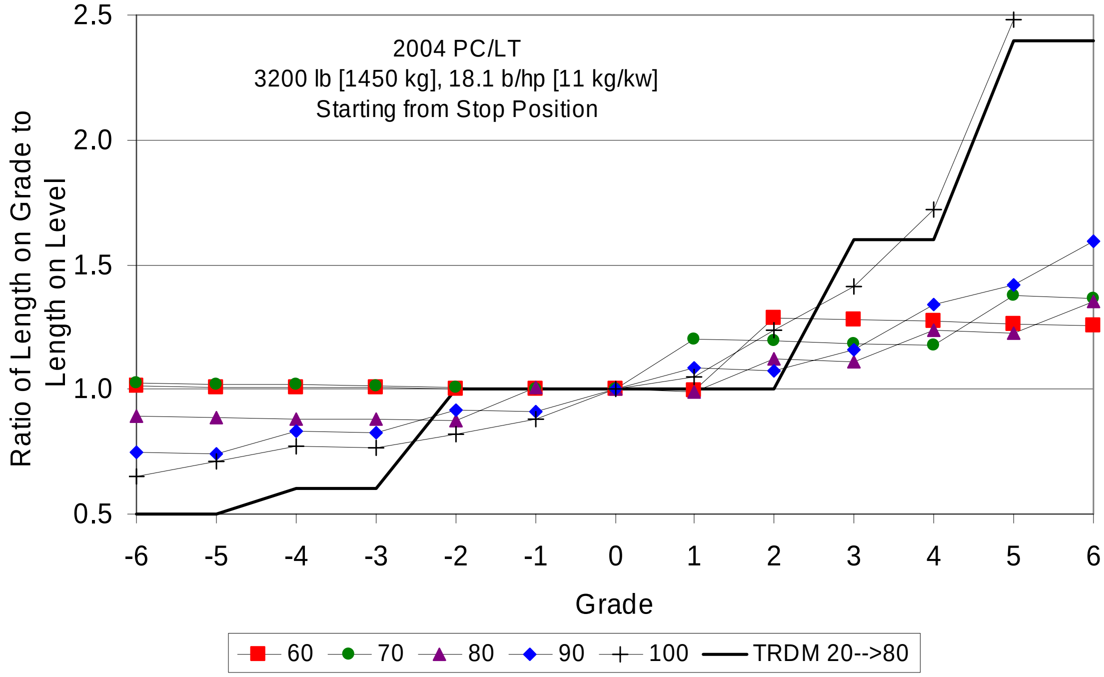
The 5544 spreadsheet has the capability of generating acceleration distances on grades. These distances could be used to determine potential adjustment factors for entrance terminals. While the distances may represent optimal acceleration conditions rather than the distances preferred by drivers when merging onto a limited-access facility, the ratios can provide an indication of potential adjustments for grades. Figure 15-14 shows the distances traveled for a 2004 PC/LT on grades ranging between -6 and 6 percent. For highway speeds of 50 mph [81 km/h] and less, the distance traveled for all grades are similar. As can be seen in Figure 15-14, at speeds of 60 mph [97 km/h] and greater a difference in distance traveled on the various grades can be viewed.

The ratio of distance traveled on each grade to the level grade was calculated. For speeds less than 50 mph [81 km/h], the ratio was near 1.0 for each situation. Above 60 mph [97 km/h] the ratio varied depending upon the final speed reached and the grade as shown in Figure 15-15. Because the TRDM/2004 *Green Book* provides adjustment factors by initial speed (i.e., ramp curve speed) as well as by speed reached (i.e., highway speed), the ratios for different speed combinations were checked to see if different patterns would result. Figure 15-16 shows the data for when the initial speed (i.e., the speed on the ramp) is 50 mph [81 km/h] rather than from a stop position. The patterns for the ratio have some differences, but overall the same trends can be observed. Table 15-14 lists potential adjustment factors developed by the research team using the data from the above examples and engineering judgment to result in logical and smooth curves (e.g., adjustment factors were higher for the higher design speeds). Figure 15-17 shows a plot of the potential adjustment factors.



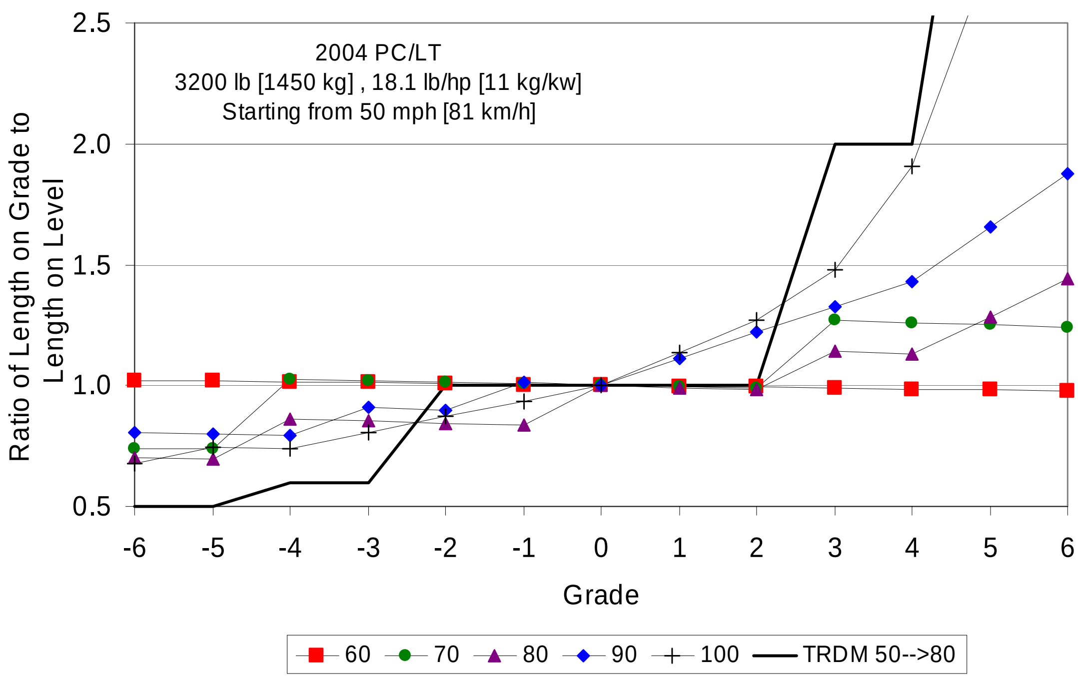
Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 15-14. Speed Distance Plot for 2004 Vehicle for Grades from -6 to 6 Percent.



Note: 1 mph = 1.61 km/h

Figure 15-15. Ratios for Grades from -6 to 6 Percent for Vehicles Accelerating from Stop Position.

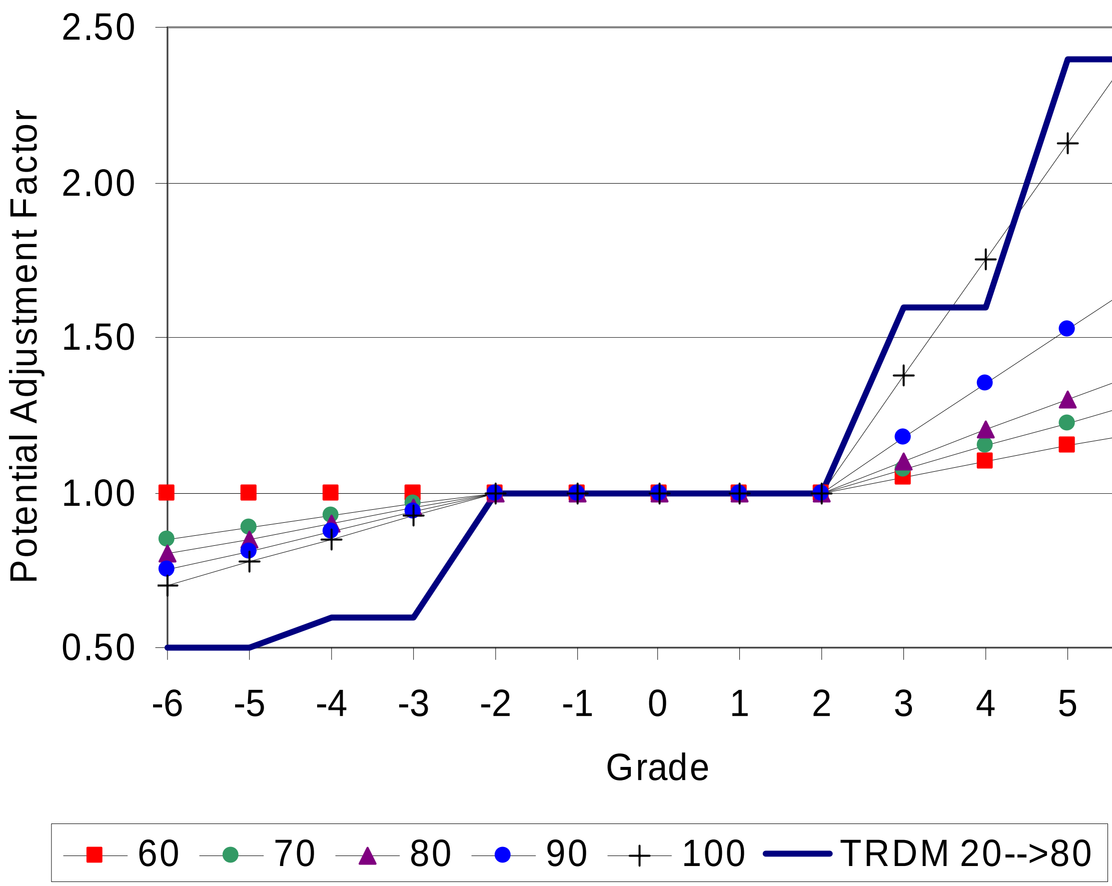


Note: 1 mph = 1.61 km/h

Figure 15-16. Ratios for Grades from -6 to 6 Percent for Vehicles Accelerating from 50 mph [81 km/h].

Table 15-14. Potential Adjustment Factors for Passenger Car/Light Truck Vehicles for Acceleration Lanes.

Highway Design Speed (mph [km/h])	-6	-5	-4	-3	-2 to 2	3	4	5	6
50 [81] and below	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
60 [97]	1.00	1.00	1.00	1.00	1.00	1.05	1.10	1.15	1.20
70 [113]	0.85	0.89	0.93	0.96	1.00	1.08	1.15	1.23	1.30
80 [129]	0.80	0.85	0.90	0.95	1.00	1.10	1.20	1.30	1.40
90 [145]	0.75	0.81	0.88	0.94	1.00	1.18	1.35	1.53	1.70
100 [161]	0.70	0.78	0.85	0.93	1.00	1.38	1.75	2.13	2.50



Note: 1 mph = 1.61 km/h

Figure 15-17. Plot of Potential Adjustment Factors for Grades from -6 to 6 Percent for Acceleration Lanes.

CHAPTER 16

ROADSIDE CLEAR ZONES

CURRENT GUIDANCE

The TRDM states the following in Chapter 2, Section 6 – Cross Sectional Elements:

> "A clear recovery area, or horizontal clearance, should be provided along high-speed rural highways. Such a recovery area should be clear of unyielding objects where practical or shielded by crash cushions or barrier" (*1*).

Table 2-11 of the TRDM (shown here as Table 16-1) specifies minimum and desirable horizontal clearance width for different roadway functional classes based on design speed and average daily traffic. The minimum clear zone specified for rural freeways is 30 ft [9.0 m] regardless of design speed or ADT.

The 2002 AASHTO *Roadside Design Guide* (*74*) provides clear zone guidance based on traffic volume, design speed, and roadside geometry. This guidance is presented in both graphical and tabular form as reproduced in Figure 16-1 and Table 16-2, respectively. It can be seen from Table 16-2 that the recommended clear zone distance is 30 to 34 ft [9 to 10.5 m] for a design speed of 65 to 70 mph [110 km/h] and an ADT over 6000 vehicles, and 1V:6H side foreslope. A footnote to Table 16-2 states that "Clear zones may be limited to 30 ft [9.0 m] for practicality and to provide a consistent roadway template if previous experience with similar projects or designs indicates satisfactory performance." A designer may choose to modify the basic clear zone distance for horizontal curvature using adjustment factors that range from 1.1 to 1.5 based on radius of curvature and design speed.

Table 16-1. Reproduction of TRDM Table 2-11: Horizontal Clearances[a] (*1*).

Location	Functional Classification	Design Speed (mph)	ADT[b]	Horizontal Clearance Width (ft)[c,d,e]	
				Minimum	Desirable
Rural	Freeways	All	All	30 (16 for ramps)	
Rural	Arterial	All	0-750 750-1500 >1500	10 16 30	16 30 --
Rural	Collector	≥50	All	Use above rural arterial criteria	
Rural	Collector	≤45	All	10	--
Rural	Local	All	All	10	--
Suburban	All	All	<8000	10 [f]	10 [f]
Suburban	All	All	8000-12000	10 [f]	20 [f]
Suburban	All	All	12000-16000	10 [f]	25 [f]
Suburban	All	All	>16000	20 [f]	30 [f]
Urban	Freeways	All	All	30 (16 for ramps)	
Urban	All (curbed)	≥50	All	Use above suburban criteria insofar as available border width permits	
Urban	All (curbed)	≤45	All	1.5 from curb face	3.0
Urban	All (uncurbed)	≥50	All	Use above suburban criteria	
Urban	All (uncurbed)	≤45	All	10	--

[a] Because of the need for specific placement to assist traffic operations, devices such as traffic signal supports, railroad signal/warning device supports, and controller cabinets are excluded from horizontal clearance requirements. However, these devices should be located as far from the travel lanes as practical. Other non-breakaway devices should be located outside the prescribed horizontal clearances or these devices should be protected with barrier.

[b] Average ADT over the life of the project, i.e., 0.5 (present ADT plus future ADT). Use total ADT on two-way roadways and directional ADT on one-way roadways.

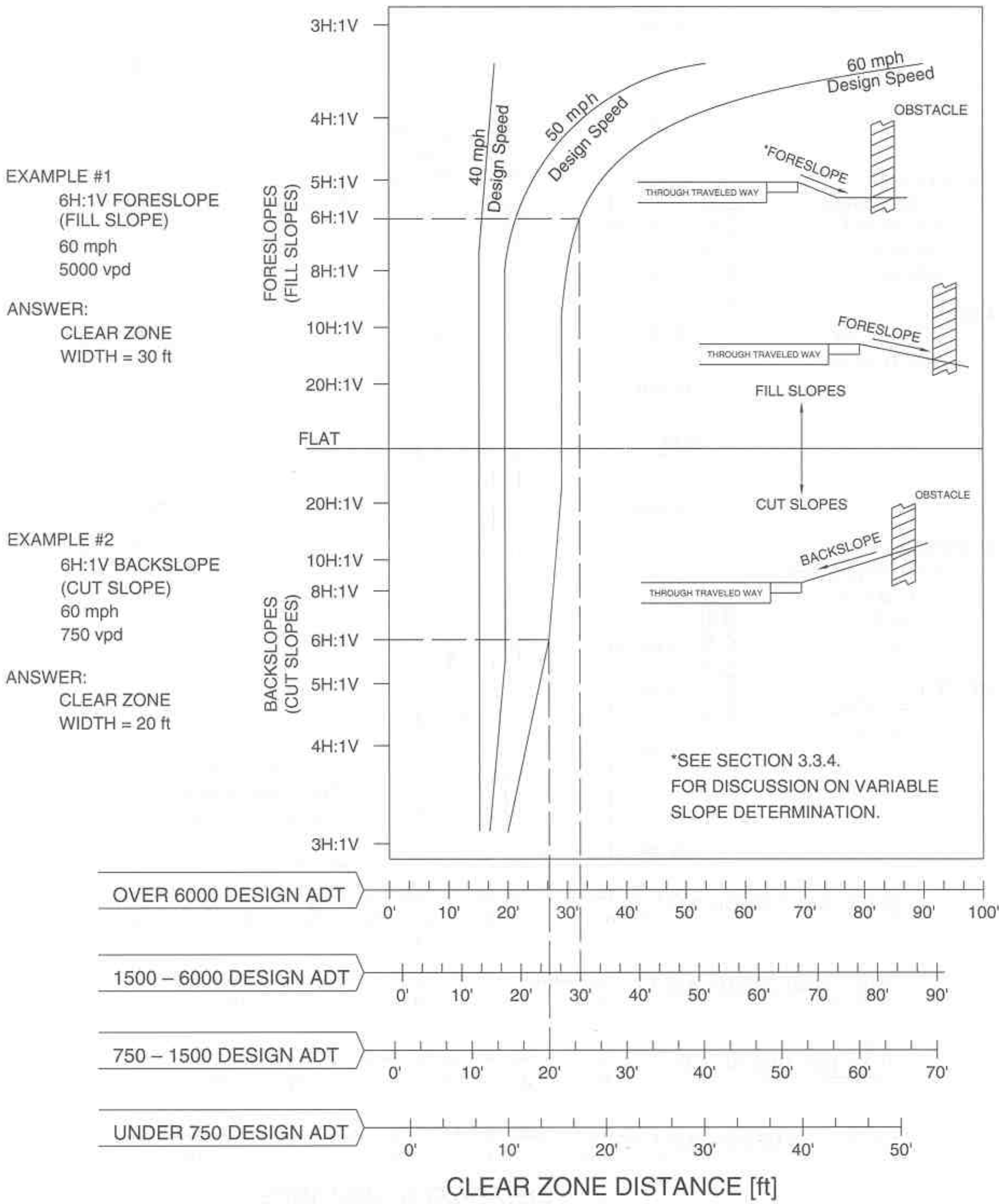
[c] Without barrier or other safety treatment of appurtenances.

[d] Measured from edge of travel lane for all cut sections and for all fill sections where side slopes are 1V:6H or flatter. Where fill slopes are steeper than 1V:6H it is desirable to provide an area free of obstacles beyond the toe of the slope.

[e] Desirable, rather than minimum, values should be used where feasible.

[f] Purchase of 5 ft or less of additional right-of-way strictly for satisfying horizontal clearance provisions is not required.

Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 16-1. Clear Zone Distances Curves (*Roadside Design Guide*, Figure 3.1b) (74).

Table 16-2. Clear Zone Distances from Edge of Traveled Way (*Roadside Design Guide*, Table 3.1) (*74*).

[U.S. Customary Units]

DESIGN SPEED	DESIGN ADT	FORESLOPES			BACKSLOPES		
		1V:6H or flatter	1V:5H TO 1V:4H	1V:3H	1V:3H	1V:5H TO 1V:4H	1V:6H or flatter
40 mph or less	UNDER 750	7 – 10	7 – 10	**	7 – 10	7 – 10	7 – 10
	750 – 1500	10 – 12	12 – 14	**	10 – 12	10 – 12	10 – 12
	1500 – 6000	12 – 14	14 – 16	**	12 – 14	12 – 14	12 – 14
	OVER 6000	14 – 16	16 – 18	**	14 – 16	14 – 16	14 – 16
45–50 mph	UNDER 750	10 – 12	12 – 14	**	8 – 10	8 – 10	10 – 12
	750 – 1500	14 – 16	16 – 20	**	10 – 12	12 – 14	14 – 16
	1500 – 6000	16 – 18	20 – 26	**	12 – 14	14 – 16	16 – 18
	OVER 6000	20 – 22	24 – 28	**	14 – 16	18 – 20	20 – 22
55 mph	UNDER 750	12 – 14	14 – 18	**	8 – 10	10 – 12	10 – 12
	750 – 1500	16 – 18	20 – 24	**	10 – 12	14 – 16	16 – 18
	1500 – 6000	20 – 22	24 – 30	**	14 – 16	16 – 18	20 – 22
	OVER 6000	22 – 24	26 – 32 *	**	16 – 18	20 – 22	22 – 24
60 mph	UNDER 750	16 – 18	20 – 24	**	10 – 12	12 – 14	14 – 16
	750 – 1500	20 – 24	26 – 32 *	**	12 – 14	16 – 18	20 – 22
	1500 – 6000	26 – 30	32 – 40 *	**	14 – 18	18 – 22	24 – 26
	OVER 6000	30 – 32 *	36 – 44 *	**	20 – 22	24 – 26	26 – 28
65–70 mph	UNDER 750	18 – 20	20 – 26	**	10 – 12	14 – 16	14 – 16
	750 – 1500	24 – 26	28 – 36 *	**	12 – 16	18 – 20	20 – 22
	1500 – 6000	28 – 32 *	34 – 42 *	**	16 – 20	22 – 24	26 – 28
	OVER 6000	30 – 34 *	38 – 46 *	**	22 – 24	26 – 30	28 – 30

*Where a site specific investigation indicates a high probability of continuing crashes, or such occurrences are indicated by crash history, the designer may provide clear-zone distances greater than the clear zone shown in Table {16-2}. Clear zones may be limited to 30 ft [9.0 m] for practicality and to provide a consistent roadway template if previous experience with similar projects or designs indicates satisfactory performance.
**Since recovery is less likely on the unshielded, traversable 1V:3H slopes, fixed objects should not be present in the vicinity of the toe of these slopes. Recovery of high-speed vehicles that encroach beyond the edge of the shoulder may be expected to occur beyond the toe of slope. Determination of the width of the recovery area at the toe of slope should take into consideration right-of-way availability, environmental concerns, economic factors, safety needs, and crash histories. Also, the distance between the edge of the through traveled lane and the beginning of the 1V:3H slope should influence the recovery area provided at the toe of slope. While the application may be limited by several factors, the foreslope parameters that may enter into determining a maximum desirable recovery area are illustrated in Figure {16-1}.
Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

DISCUSSION

Until the 1960s, little emphasis was placed on roadside safety design. The prevailing philosophy was that reasonable and prudent drivers did not inadvertently leave the travelway and the penalty for doing so by others was acceptable. Studies by Stonex at the General Motors (GM) Proving Ground in the late 1950s and early 1960s (*75, 76*) showed that even professionally trained drivers strayed from the travelway and that measures to minimize risks of roadside encroachments were both desirable and warranted. Work at the GM Proving Ground contributed significantly to acceptance by many of the need for a "forgiving roadside." This need was underscored by the alarming number of run-off-the-road, single-vehicle crashes and the high severity associated with these crashes.

Results of the GM studies also formed the basis for initial dimensions of recommended "recovery areas" (*77*, *78*). These areas were later referred to as "clear zones" (*79*), "clear recovery zones" (*80*), roadside recovery distance (*81*), or horizontal distance. The GM studies provided probability data on lateral extent of vehicular movement for run-off-the-road crashes. Using these data, AASHO and subsequently AASHTO suggested that, where feasible, a clear, unencumbered recovery area should extend 30 ft [9.1 m] or more laterally from the travelway (*78*, *79*). The GM studies indicated that the lateral extent of vehicular movement would not exceed 30 ft [9.1 m] in approximately 80 percent of run-off-the-road crashes on high-speed highways.

This 30-ft [9.1 m] clear zone value was incorporated into the second edition of AASHTO's *Highway Design and Operational Practices Related to Highway Safety* (a.k.a. the *Yellow Book*) (*77*), which was published in 1974. The *Yellow Book* also stated that "for adequate safety, it is desirable to provide an unencumbered roadside recovery area that is as wide as practical on a specific highway section." Subsequently, many highway agencies adopted a 30-ft [9.1 m] clear recovery area beyond the edge of the traveled way.

National guidelines continued to recommend a 30-ft [9.1 m] clear zone until 1977. However, the 30-ft [9.1 m] width was recognized as being somewhat arbitrary, because it was based on crash studies at the GM Proving Grounds where relatively flat roadsides were provided. The 1977 AASHTO *Barrier Guide* (*79*) contained clear zone recommendations that were dependent on design speed, the slope of the cut or fill section, and whether or not there was rounding at the hinge at the juncture of the shoulder with the sideslope. These guidelines indicated that the width of the clear zone should increase with increasing design speed and increasing steepness of fill slopes. For example, the recommended clearance for a high-speed roadway (60-mph [97 km/h] design speed) with a fill section having a 1V:4H unrounded sideslope was approximately 43 ft [13 m]. For the same example and a 40-mph [64 km/h] design speed, the recommended clearance was approximately 18 ft [5.5 m].

Clear zone criteria contained in the 1977 AASHTO *Barrier Guide* (*77*) were developed by Ross et al. in a research study (*82*) sponsored by the Federal Highway Administration. In the study, the Highway Vehicle Object Simulation Model (HVOSM) computer program (*83*) was used to determine the lateral extent of vehicular movement for encroachments on fill and cut roadside sections, rounded and unrounded, at speeds of 40, 50, and 60 mph [64, 81 and 97km/h]. Assumed driver responses for the simulated encroachments included an emergency steer-back-to-the-travelway maneuver and emergency full braking.

The 1989 AASHTO *Roadside Design Guide* (*84*) contained certain revisions to the clear zone criteria of the 1977 AASHTO *Barrier Guide*. In addition to the variables considered in the 1977 *Barrier Guide*, clear zone widths were also defined in terms of traffic volume. Greater ranges of design speed were adopted, but the effects of slope rounding were not considered. Clear zone criteria presented in the 1989 *Roadside Design Guide* were derived from data in the 1977 *Barrier Guide*, in combination with state practices and the collective judgment of the task force that prepared the *Guide*. The following statement in the 1989 *Roadside Design Guide* reminded users of the subjective nature of the clear zone recommendations and that engineering judgment was essential in their application:

> "...The numbers obtained from Figure 3.1 or Table 3.1 imply a degree of accuracy that does not exist. Again, the curves are based on limited empirical data which was then extrapolated to provide data for a wide range of conditions. Thus, the numbers obtained from these curves represent a reasonable measure of the degree of safety suggested for a particular roadside, they are neither absolute nor precise..." *(84)*.

The revisions to clear zone guidance adopted in the 1989 *Roadside Design Guide* have remained essentially unchanged through the current 2002 edition of the *Roadside Design Guide (74)*.

Current guidance provided in the 2002 AASHTO *Roadside Design Guide* recommends a clear zone distance of 34 ft [10.4 m] for highways with a design speed of 70 mph [113 km/h], design ADT over 6000 veh/day, and foreslopes of 1V:6H or flatter *(74)*. Potential clear zone design values for high design speeds were extrapolated from this value using two different methodologies.

The first method is based on the reaction time afforded the driver of an errant vehicle under current practice. Given a prescribed clear zone distance (e.g., 34 ft [10.4 m]), an assumed angle of encroachment, and a given encroachment speed, the reaction time available to a motorist can be determined. If encroachment speed is assumed to remain proportional to design speed, a clear zone distance that provides the same driver reaction time as current guidance can be computed for different values of design speed. As shown in Figure 16-2, the extrapolated clear zones derived from driver reaction time vary linearly with design speed and are independent of the assumed encroachment angle. Based on this method of extrapolation, the clear zone distance associated with a 100-mph [160 km/h] design speed is approximately 48 ft [14.6 m].

While driver reaction time varies linearly with encroachment speed, the vehicle response does not. The second extrapolation method takes into account the response of the vehicle, namely braking distance. Braking distance can be computed as a function of encroachment velocity, V_i, and the coefficient of friction between the tires and roadside surface, n, as shown in the equation below:

$$x_f - x_i = \frac{1}{2}\frac{V_i^2}{ng} \qquad \textbf{(16-1)}$$

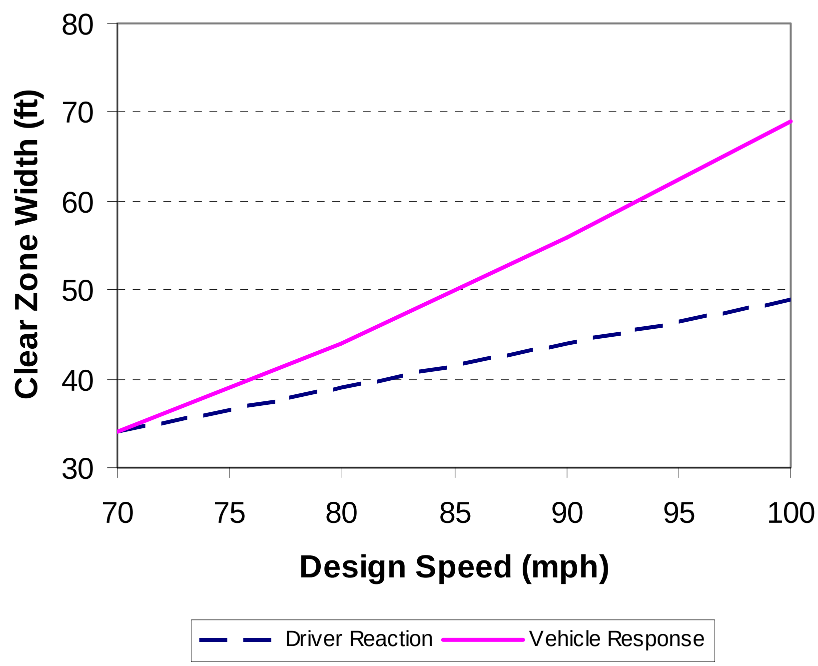
Where:

X_i = initial position, ft;
X_f = final position, ft;
V_i^2 = encroachment velocity, ft/s^2;
n = coefficient of friction between the tires and roadside survey; and
g = gravity constant.

As illustrated by this equation, braking distance varies with the square of the encroachment velocity. Therefore, a clear zone value extrapolated on the basis of driver reaction time will not provide a stopping ability proportional to that provided by current guidance.

Based on prescribed encroachment speed and assumed coefficient of friction (e.g., 0.5 for dry grassy surface), a braking distance can be computed. Given this braking distance and a prescribed lateral clear zone (e.g., 34 ft [10.4 m]), the maximum encroachment angle at which a vehicle can be brought to a stop under current clear zone guidance can be calculated. For a 34-ft [10.4 m] clear zone and a speed of 70 mph [113 km/h], this encroachment angle is approximately 6 degrees.

Assuming encroachment speed remains proportional to design speed, the encroachment angle computed under current guidance and a braking distance computed for a higher design speed can be used to compute a lateral clear zone distance required to bring an errant vehicle to a stop. The clear zone relationship resulting from an extrapolation based on braking distance is shown in Figure 16-2. Based on this method of extrapolation, the clear zone distance associated with a 100-mph [160 km/h] design speed is approximately 70 ft [21.4 m].



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 16-2. Extrapolated Clear Zone Distances for High Design Speeds.

POTENTIAL CLEAR ZONE VALUES FOR HIGH DESIGN SPEEDS

The clear zone is the fundamental cornerstone of the "forgiving roadside" concept. TRDM states that "For adequate safety, it is desirable to provide an unencumbered roadside recovery area that is as wide as practical for the specific highway and traffic conditions" (*1*). The increased severity of impacting a roadside hazard at high speeds will result in a higher probability of serious and fatal injuries. For these reasons, it is recommended that the clear zone values derived from the vehicle response extrapolation in Figure 16-2 be used for controlled-access facilities with design speeds over 80 mph [137 km/h].

RESEARCH NEEDS

Current clear zone guidelines are based on a very limited number of vehicle encroachment simulations conducted in the 1970s and engineering judgment. The extrapolation of these guidelines to high design speeds is therefore tenuous. Unfortunately, relationships between various vehicle, roadway, and roadside variables and lateral extent of encroachment have not been fully established. The use of crash data for determining the statistics on the extent of lateral movement of vehicles encroaching onto the roadside is limited because the lateral extent of encroachment in roadside crashes is controlled by the lateral offset of the object struck.

Under NCHRP Project 17-11 (*85*), TTI researchers utilized computer simulation to overcome these crash data limitations. The computer simulation approach permits a detailed analysis of vehicle trajectory and resulting vehicle kinematics for a wide range of variables for which data may not otherwise be available. The resulting data can be used to determine the influence of and develop relationships between various encroachment parameters that could not otherwise be studied. A similar approach, as outlined below, can be used to develop more definitive clear zone guidance for high-speed highways.

An extensive computer simulation study using a vehicle dynamics code can be conducted to determine lateral extent of movement for vehicular encroachments across various roadside slopes and ditches. The variables that can be considered in the computer simulation study include:

- vehicle type,
- encroachment speed and angle,
- vehicle orientation,
- driver input,
- horizontal curvature,
- shoulder width,
- foreslope ratio, and
- ditch configuration (e.g., foreslope width, ditch width, backslope ratio, backslope width).

To use the results from discrete simulations to infer the encroachment characteristics of the general vehicle-crash population, known or estimated distributions can be applied to each encroachment parameter to develop "expansion factors" or probabilities for each of the encroachment variable categories used in the simulation matrix. The probability for vehicle type can be derived from vehicle sales data. Probability distributions for other key encroachment parameters (e.g., encroachment speed, encroachment angle, vehicle orientation at encroachment [i.e., tracking or non-tracking], and driver control input [i.e., braking and/or steering]) can be determined based on crash data derived from NCHRP Project 17-22 (*86*). Under Project 17-22, run-off-road crash cases were selected from the National Automotive Sampling System (NASS) Crashworthiness Data System (CDS) for supplemental field data collection, reconstruction, and clinical analysis. The purpose of the clinical analysis is to develop a database for ran-off-road crashes from which distributions for different characteristics of these crashes (including encroachment speed and angle) can be derived.

The encroachment speed and angle distributions derived from crash data on 65- to 75-mph [105 to 121 km/h] speed limit roadways will need to be extrapolated to high-speed (e.g., 80 to 100 mph [137 to 160 km/h]) roadways. This extrapolation will be done using a parametrical (statistical) regression model. Current thinking is to use gamma regression models with an exponential link function similar to those developed by TTI researchers under NCHRP Project 17-11 (*85*). Explanatory variables included in these regression models are posted speed limit, number of lanes, land use (urban/rural), and median type (divided vs. undivided). The quality of the extrapolation will depend on the size of data sample available at the 65- to 75-mph [105 to 121 km/h] speed limit range and the extent of the extrapolation to be made.

These extrapolated distributions can be aggregated to obtain input probabilities for the value categories used in the simulation matrix. The combined probability for a given simulation with a unique set of encroachment conditions will then be determined by multiplying the individual probabilities assigned to the value of each encroachment parameter. The probability that a vehicle encroaching onto the roadside will have a lateral extent of movement within a specified range is simply the sum of the probabilities of the simulated encroachments that have a maximum extent of lateral movement within that range. In this way, exceedance curves will be derived for lateral extent of movement associated with roadside encroachments for a given set of roadway and roadside conditions.

Clear zone guidance can be developed from these relationships in one of two ways: by establishing some criteria or threshold (e.g., 85th percentile) for clear zone distance from the exceedance curves, or by using the encroachment relationships in a benefit-cost analysis. In this manner, well-founded clear zone guidance can be established for high-speed facilities.

CHAPTER 17

MEDIAN WIDTH

CURRENT GUIDANCE

According to the TRDM (*1*), longitudinal concrete barriers in medians are provided to prevent:

- "unlawful turns,
- out-of-control vehicles from entering the opposing traffic lanes, and, in some cases
- unlawful crossing of medians by pedestrians."

Guidance for median barriers is differentiated on the basis of control of access and median width. Median barriers are generally provided for controlled-access highways with medians of 30 ft [9.0 m] or less in width. Median barriers *may* be provided for non-controlled-access highways with similar medians, but their use should generally "be restricted to areas with potential safety concerns such as railroad separations or through areas where median constriction occurs." If justified through an operational analysis, median barriers may be provided for medians with widths greater than 30 ft [9.0 m]. Typical freeway sections are shown in Figure 17-1, indicating median barrier placement for freeways with median widths less than or equal to 30 ft [9.0 m].

Other uses for concrete median barriers include preventing vehicles from striking hazardous obstacles or encountering steep slopes. Guidance for this application would be derived from design charts and tables in Appendix A of the TRDM. Other sections of the TRDM discuss design considerations for median barriers such as the potential for introducing a sight restriction on horizontal curves, the need for periodic openings to provide emergency vehicle access, and the need to adequately treat the endpoints of barriers (*1*). Design details are further supplemented in standard design drawings available from TxDOT's Design Division. These standards provide barrier construction details and guidance regarding the safety treatment of median barrier ends.

The AASHTO *Roadside Design Guide* (*74*) defines a median as that portion of a divided roadway, including the inside shoulders, that separates the traveled way for through traffic in opposing directions of travel. The primary function of a median is to separate opposing traffic flows to prevent a vehicle from crossing the median and becoming involved in a head-on crash. However, other functions served by roadway medians include:

- providing a recovery area for errant vehicles;
- a stopping location and refuge area for emergency situations;
- allowing space for changes in vehicle speed and storage of left-turning and u-turning vehicles;
- minimizing glare from on-coming headlights; and
- providing width for future expansion of the travel lanes.

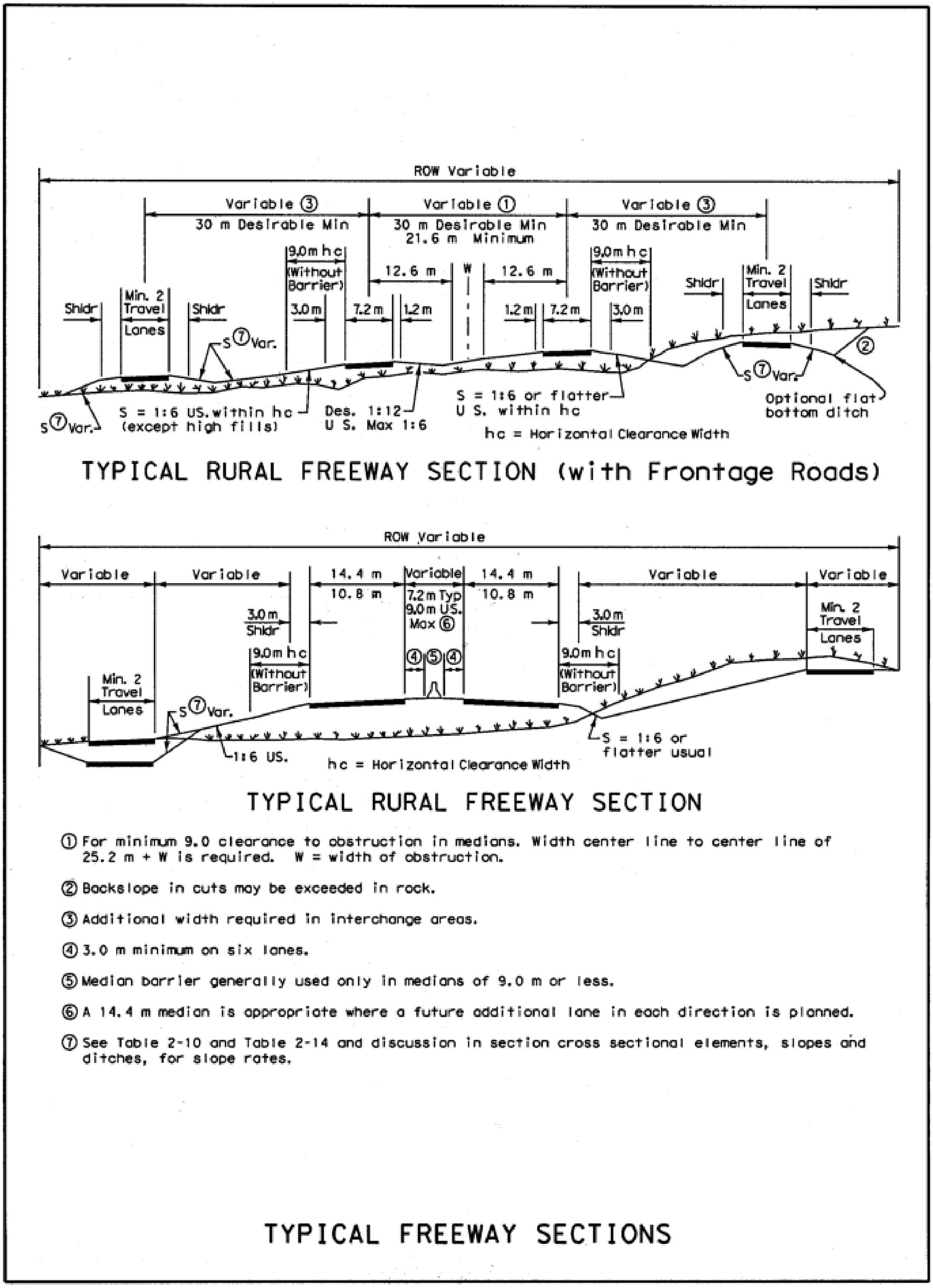


Figure 17-1. Typical Freeway Section (Figure 3-12 from TRDM) (*1*).

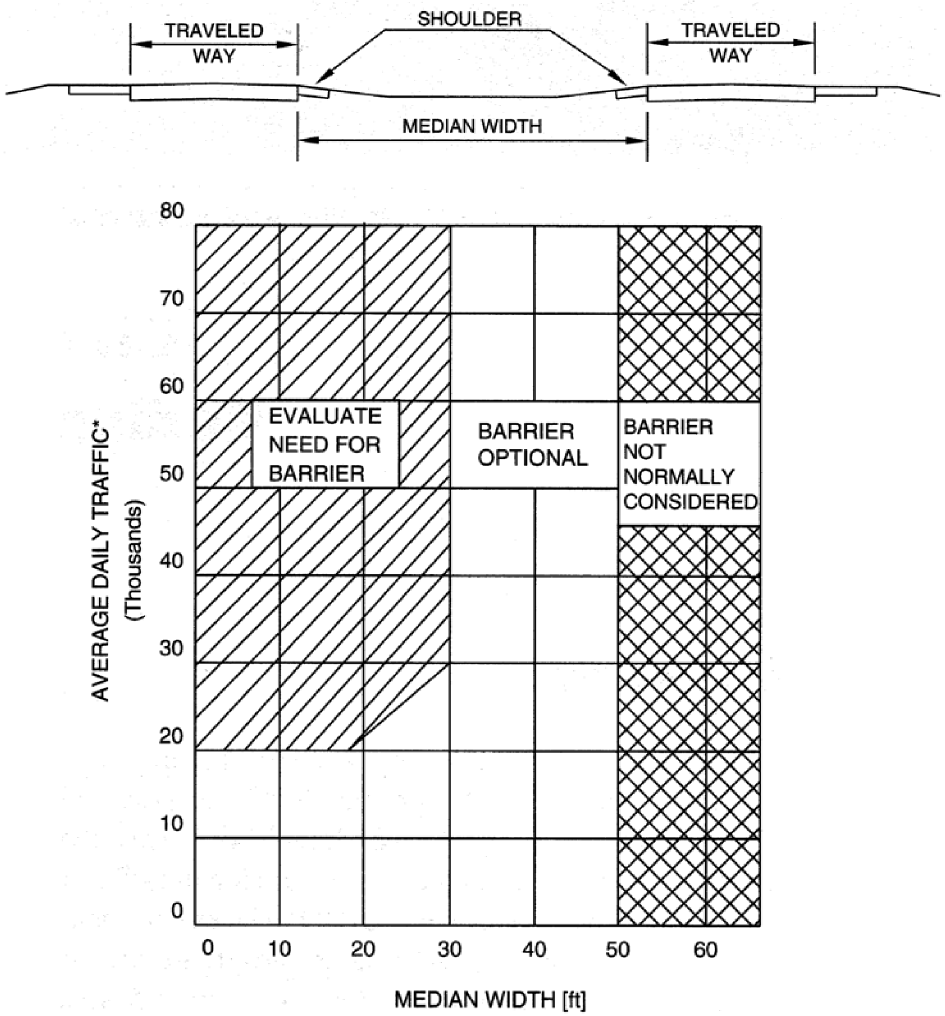
For high-speed, controlled-access roadways that have relatively flat, traversable medians, AASHTO's guideline indicates that the designer should evaluate the need for barrier on all medians up to 30 ft [9.0 m] in width when ADT is 20,000 vehicles per day (vpd) or greater. Barrier is optional for all medians with width between 30 ft [9.0 m] and 50 ft [15.3 m] or when the median width is less than 30 ft [9.0 m] and the average annual daily traffic (AADT) is less than 20,000 vpd.

Chapter 6 of *Roadside Design Guide* has been revised recently based largely on cross-median crash research by various state departments of transportation. The revised median barrier guidance for divided roadways with ADT greater than 20,000 vpd is shown in Table 17-1. The median width for which median barrier is typically considered has been extended from 30 ft [9.0 m] to 50 ft [15.3 m] and the range for which barrier application is optional has been extended to 70 ft [21.4 m].

Table 17-1. Revised Median Barrier Installation Guidance for ADT Greater Than 20,000 vpd.

Median Width (ft, m)	Barrier Installation Guidance
0 – 30 [0 – 9.1]	Barrier Recommended
30 – 50 [9.1 – 15.3]	Barrier Considered
50 – 70 [15.3 – 21.4]	Barrier Optional

Some differences are apparent between *Roadside Design Guide* and TRDM. *Roadside Design Guide* includes consideration of traffic volume in the median barrier selection decision process (see Figure 17-2) whereas TRDM does not. While the guidelines mention crash history as a median barrier warranting criterion there is no guidance given on specific cross-median crash rates that might justify the use of median barrier.



Note: 1 ft = 0.305 m

Figure 17-2. Suggested Guidelines for Median Barriers on High-Speed Roadways (*Roadside Design Guide*, Figure 6.1) (*74*).

OTHER GUIDANCE

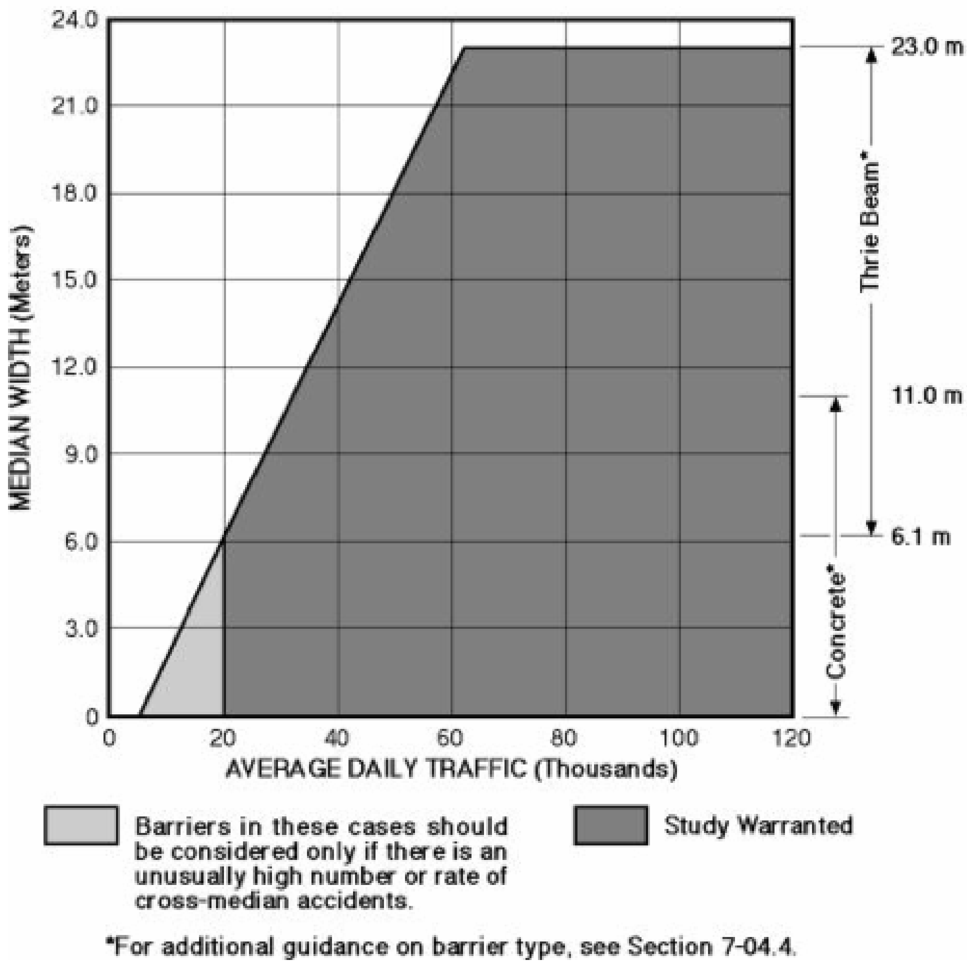
Several ongoing and recently completed research projects have been conducted at the state and national level that provide valuable insight in regard to appropriate median widths. While the majority of states still rely on the *Roadside Design Guide* as the basis for their median barrier guidelines, there are a growing number of states (including Arizona, California, Georgia, Missouri, Nevada, North Carolina, Pennsylvania, and Washington, among others) that have extended their practices to include treatment of wider median sections based on a specified cross-median crash rate threshold or some form of cost-effectiveness analysis. A brief review of some selected studies is provided below.

In 1998, North Carolina Department of Transportation (NCDOT) implemented a more stringent policy of installing median barriers for all new construction, reconstruction, and resurfacing projects with median width of 70 ft [21.4 m] or less on freeways. It also included a traffic improvement program to install cable median barriers on approximately 1000 miles [1600 km] of freeways over the 2000-2006 time period (*87*). While in the last two decades or so median barriers have either been metal-beam guardrail (including W-beam and thrie beam) or concrete barrier (e.g., New-Jersey and constant slope), NCDOT was one of the first states to use cable barriers in recent years (*88*).

In 1998, California Department of Transportation (Caltrans) also adopted more stringent guidelines based on AADT for freeways with median width less than 75 ft [22.9 m] (*89*). Concrete barriers are recommended for medians less than 20 ft [6.1 m] in width; concrete and thrie beam barriers can both be used for medians between 20 and 36 ft [6.1 m and 11.0 m], and thrie beam barriers are recommended for medians 36 to 75 ft [11.0 m to 22.9 m] in width. A crash-history warrant was also developed justifying further analysis to determine the advisability of a barrier when a site exceeds 0.5 cross-median crashes of any severity level per mile per year [0.8 cross-median crashes of any severity level per km per year] or 0.12 fatal cross-median crashes per mile per year [0.19 fatal cross-median crashes per km per year]. Figure 17-3, taken from the Caltrans *Traffic Manual*, shows a graphical version of the Caltrans volume/median width warrant (*90*).

Glad et al. (*91*) reports a benefit/cost (B/C) analysis conducted by Washington State Department of Transportation (WSDOT) for cable, metal guardrail, and concrete barriers and researchers concluded that barriers placed in median sections up to 50 ft [15.3 m] wide are cost effective for high-speed (>45 mph [72 km/h] in posted speed limit), high-volume, multilane, access-controlled, and divided state highways. An in-service study was recently conducted on cable median barriers installed in the mid-1990s to analyze their initial installation cost, maintenance cost and experience, and crash history (*92*). The authors noted that while the overall number of crashes increased noticeably, the number of severe crashes (fatal and disabling) decreased significantly. In addition, they concluded that the installation of cable barriers had a societal benefit of about $420,000 per mile annually.

Florida Department of Transportation (FDOT) requires median barrier installations on interstate highways if the median width is less than 64 ft [19.5 m], and if less than 60 ft [18.3 m] and 40 ft 12.2 m], respectively, for freeways with design speed greater than or equal to and less than 60 mph [97 km/h] (*93*). FDOT also indicates that "a median barrier shall be provided … where reconstruction reduces the median width to less than the standard for the facility. No variations or exceptions to this criterion will be approved." FDOT further requires a cross-median crash history evaluation for any interstate and expressway project. The evaluation must be conducted in the area of interchanges 1 mile prior to the exit ramp gore and 1 mile beyond the entrance ramp gore. If there are three or more cross-median crashes in the most recent five-year period within that segment, median barrier shall be provided and no B/C analysis is required. Depending on the length of the weaving section, this is about 0.35 to 0.4 cross-median crashes per mile per year [0.56 to 0.64 cross-median crashes per km per year], which is more stringent than the Caltrans' crash history warrant of 0.5 crashes per mile per year [0.8 crashes per km per year]. For those roadway sections that have fewer than three cross-median crashes, a B/C analysis shall be conducted to determine the need. In addition, for the remaining area of the project (outside of the gore area indicated above), both cross-median crash history and B/C analyses need to be performed to justify the need.



Note: 1 ft = 0.305 m

Figure 17-3. California Freeway Median Barrier Warrants (*90*).

The National Highway Cooperative Research Program sponsored Project 17-14, "Improved Guidelines for Median Safety" (*94*). This study was conducted because the major documents (i.e., the *Roadside Design Guide* and *Policy on Geometric Design of Highways and Streets*) used in the design and redesign of medians are based on old data that may not reflect current conditions.

The NCHRP 17-14 project, which was conducted by BMI and the University of North Carolina (UNC) Highway Safety Research Center (HSRC), produced a draft final report in July 2004 (*94*) that was not published. Unfortunately, collection of data needed for Project 17-14 proved to be very expensive, and the data limitation hampered the strength of the recommendations. The project recommendations have not been incorporated into practice but should be very beneficial in future research.

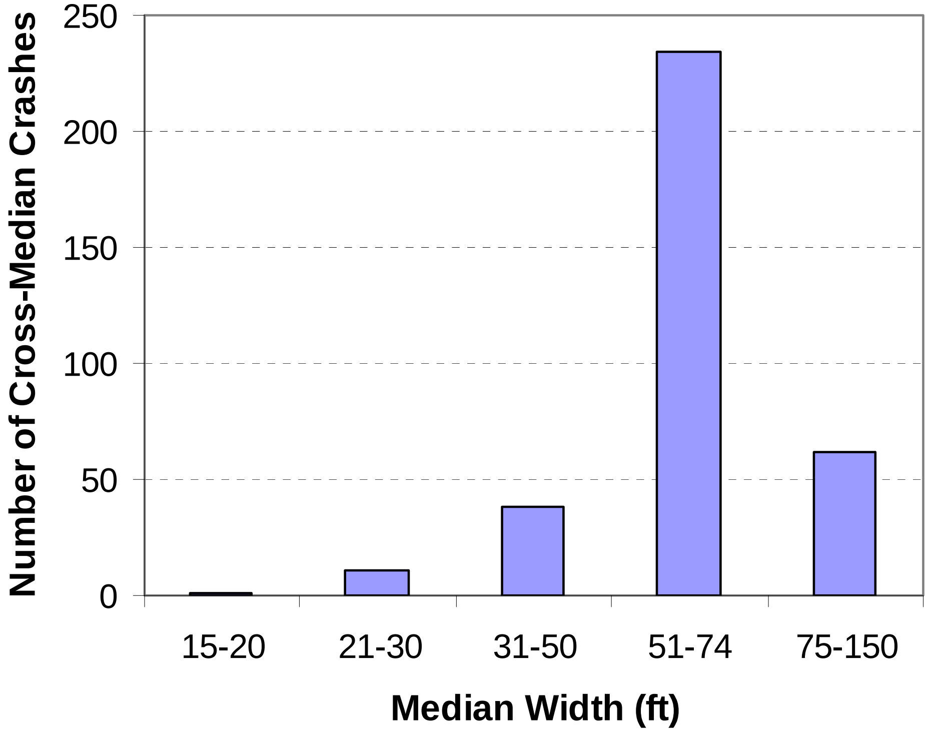
To avoid some of the obstacles that NCHRP Project 17-14 faced, NCHRP Project 22-21, *Median Cross-Section Design for Rural Divided Highways*, will focus on typical cross-section designs selected for a construction or reconstruction project rather than the exact cross-section design at a particular point *(95)*. The typical cross-section designs are determined fairly early in the design process before adjustments are made to account for variations that occur along the alignment (e.g., horizontal and vertical curves, interchanges and intersections, and special drainage requirements). Project 22-21 started in January 2006 and has a scheduled completion of January 2009.

Under TxDOT Project 0-4254 *(96)*, improved guidelines for the use of median barriers on new and existing high-speed, multilane, divided highways in Texas were developed based on a benefit-cost analysis approach. The research approach taken by the study consisted of collecting roadway and median-related crash data to support a "cross-sectional with-without" analysis. The design aimed at estimating and comparing the crash frequency and severity for two groups of sites assembled from a cross section of highways in the same time period: sites with median barriers and sites without median barriers. The highways of interest were those classified as interstates, freeways, and expressways with four or more lanes and have posted speed limits of 55 mph [89 km/h] or higher.

The distribution of sampled cross-median crashes in Texas by median width is shown in Figure 17-4. The majority of cross-median crashes (234 crashes, 67.6 percent) occurred on roads having median width between 51 ft [15.6 m] and 74 ft [22.6 m]. In addition, 62 crashes (18 percent) occurred on roads with a median width greater than 75 ft [22.9 m]. Most interstate highways and other controlled-access highways that have medians 30 ft [9.0 m] or less in width have already been treated with median barrier in accordance with current TxDOT guidelines.

Poisson-gamma regression models were used to estimate/predict the frequency of cross-median and median-related crashes and an ordered multinomial logit model was used to estimate/predict the severity distributions. These models include variables such as AADT, number of lanes, posted speed limit, median width, and so on. The maximum posted speed limit in the data was 70 mph [113 km/h].

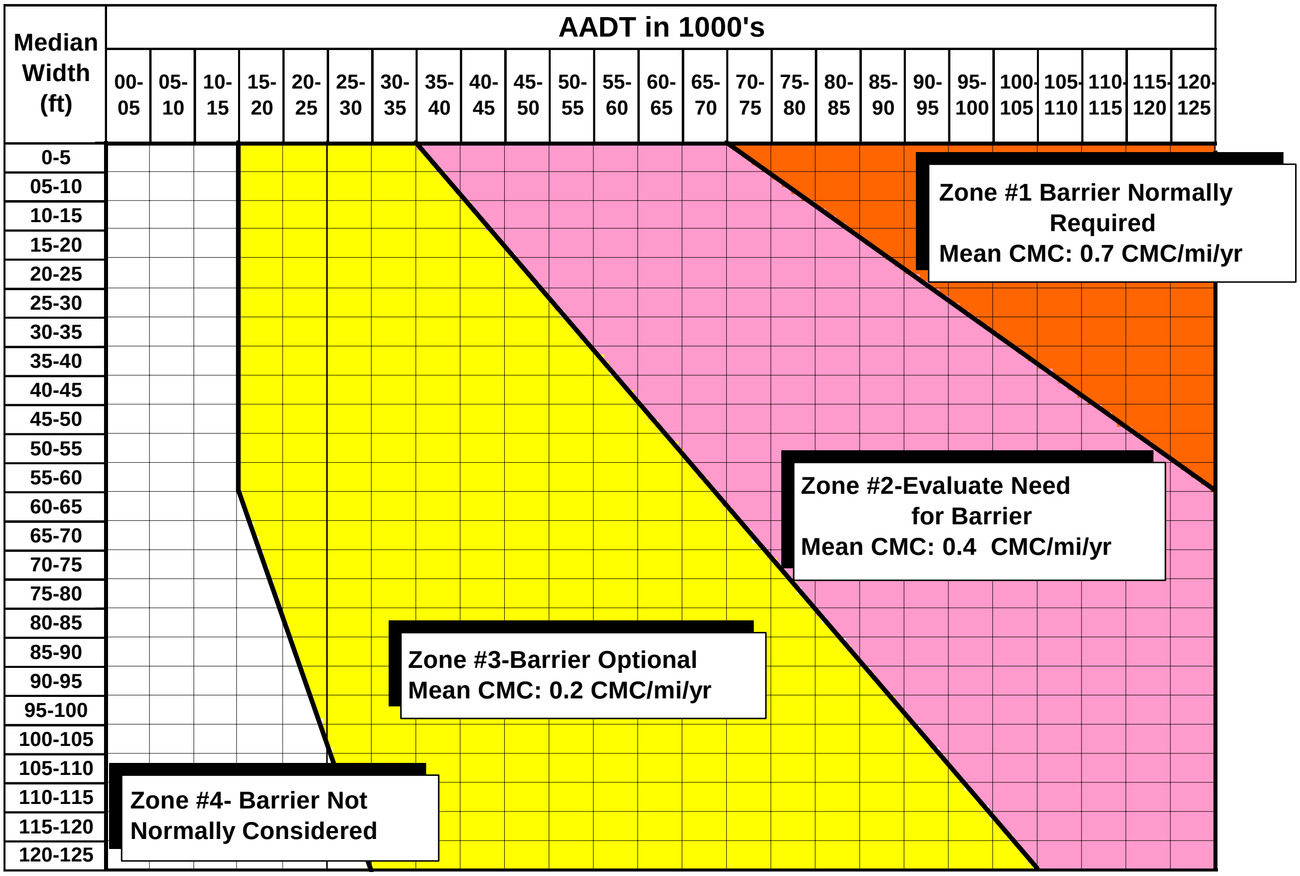
Based on the estimates from the frequency and severity models, benefit-cost ratios for installing median barriers were computed for various AADT and median-width combinations. Figure 17-5 presents a recommended guideline for installation of median barriers on high-speed, controlled-access highways in Texas that have relatively flat, traversable medians. These criteria are based on an economic analysis of median crossover crashes and other median-related crashes occurring in Texas on the selected highway classes. The guideline is divided into four different zones defined by various combinations of average annual daily traffic and median width. Each zone has an associated mean cross-median crash (CMC) rate that can be used to evaluate cross-median crash history on an existing highway section.



Note: 1 ft = 0.305 m

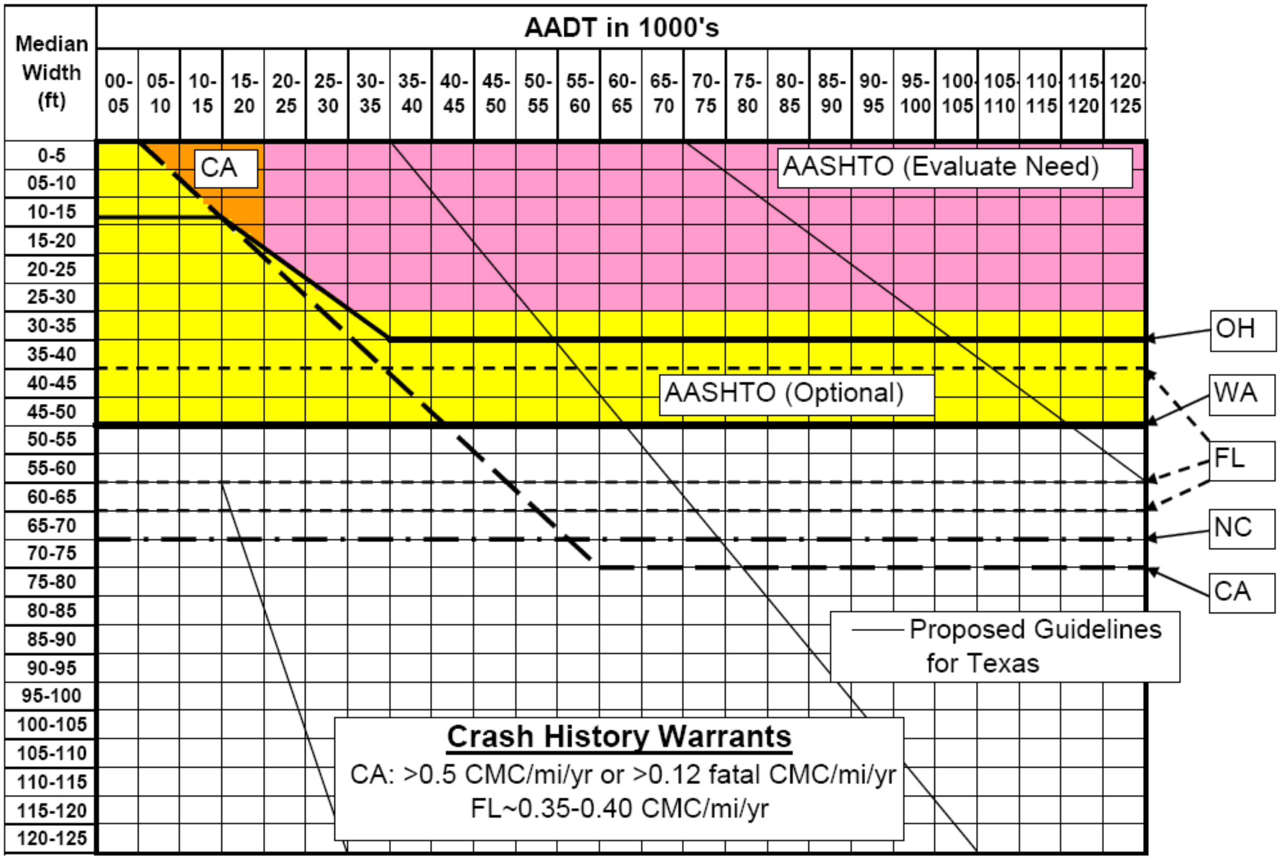
Figure 17-4. Texas Cross-Median Crash Frequency by Median Width (Including Shoulders), 1998-1999.

The various median barrier guidelines described above have been graphically plotted in Figure 17-6 for comparison with each other and with the proposed guidelines for Texas developed under TxDOT Project 0-4254. As indicated in Figure 17-6, several states (including some not shown) have adopted guidelines that require use of median barrier in medians with widths beyond those currently published by AASHTO in the 2002 *Roadside Design Guide*. Several states' guidelines (e.g., Florida, North Carolina, Washington) are based on median width without consideration of average daily traffic. These are represented as horizontal lines on Figure 17-6. Other states' guidelines (e.g., California, Ohio) vary with ADT up to some threshold median width beyond which median barriers are not required.



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 17-5. Recommended Median Barrier Guidelines for Texas (Project 0-4254) (*96*).



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 17-6. Recommended Median Barrier Guidelines Nationwide (*96*).

DISCUSSION

Many research studies have been performed to investigate the effects of median width, median barrier, and median cross-slope on cross-median crashes and overall safety. For most divided highways, the median width is already established or constrained by right-of-way restrictions. Designers often consider narrowing the median to reduce the right-of-way required. Such decisions can result in facilities that require median barrier protection along their entire length. There is a need for an analysis of the characteristics of median-related crashes and an investigation into the use of median barriers to identify changes to current standards, specifications, and procedures for median barrier need, selection, and placement that will result in the highest practical level of safety for high-speed roadways.

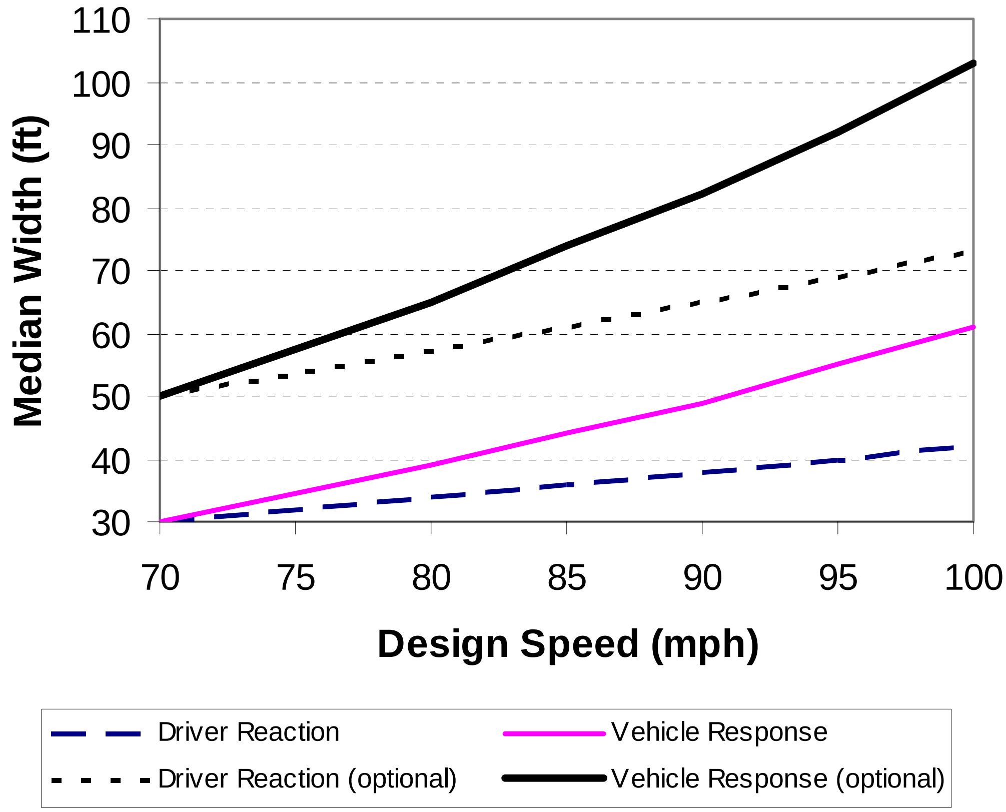
Note that the presence of a median barrier does not eliminate crashes occurring in medians but alters the character of those crashes. The construction of median barrier may actually result in an increase in total median crashes at a given location. However, a reasonable set of median barrier guidelines to help identify locations to be evaluated by an engineer for median barrier application should reduce the number of cross-median crashes. With a substantial reduction in cross-median crashes, the overall severity of median-related crashes can be significantly reduced.

For high-speed, high-volume, controlled-access roadways that have relatively flat, traversable medians, the AASHTO *Roadside Design Guide* and TxDOT *Roadway Design Manual* recommend that the need for barrier be evaluated on all medians up to 30 ft [9.0 m] in width (*1, 74*). A recently approved revision to Chapter 6 of the *Roadside Design Guide* will extend the median width for which median barrier is typically required to 50 ft [15.3 m]. As with the clear zone, design values for median widths for high design speeds were extrapolated using two different methodologies. Both the current 30-ft [9.2 m] median width and newly approved 50-t [15.3 m] median width were used as the basis for the extrapolation.

The first method is based on the reaction time afforded the driver of an errant vehicle under current practice. Given a prescribed median width and an assumed angle of encroachment, the reaction time available to a motorist can be determined for a given encroachment speed. If encroachment speed is assumed to remain proportional to design speed, a median width that provides the same driver reaction time as current guidance can be computed for different values of design speed. As shown in Figure 17-7, the extrapolated median widths derived from driver reaction time vary linearly with design speed and are independent of the assumed encroachment angle. Using a 50-ft [15.3 m] median at a design speed of 70 mph [113 km/h] as the basis for the extrapolation, Figure 17-7 indicates that the median width associated with a 100-mph [160 km/h] design speed is approximately 70 ft [21.4 m]. If 30 ft [9.0 m] is used as the current guidance, the extrapolated value for median width on facilities with a design speed of 100 mph [160 km/h] decreases to approximately 43 ft [13.1 m].

While driver reaction time varies linearly with encroachment speed, the vehicle response does not. The second extrapolation method takes into account the response of the vehicle, namely braking distance. Based on a prescribed encroachment speed and assumed coefficient of friction (e.g., 0.5 for dry grassy surface), a braking distance can be computed. Given this braking distance and a prescribed median width (e.g., 50 ft [15.3 m]), the maximum encroachment angle at which a vehicle can be brought to a stop under current median width guidance can be calculated.

Assuming encroachment speed remains proportional to design speed, the encroachment angle computed under current guidance and a braking distance computed for a higher design speed can be used to compute a median width required to bring an errant vehicle to a stop. The median width relationship resulting from an extrapolation based on braking distance is shown in Figure 17-7. Based on this method of extrapolation, the median width associated with a 100-mph [160 km/h] design speed for which barrier would not typically be required is approximately 100 ft [31 m]. If 30 ft [9.0 m] is used as the current median width guidance, the extrapolated value for median width on facilities with a design speed of 100 mph [160 km/h] decreases to approximately 70 ft [21.4 m].



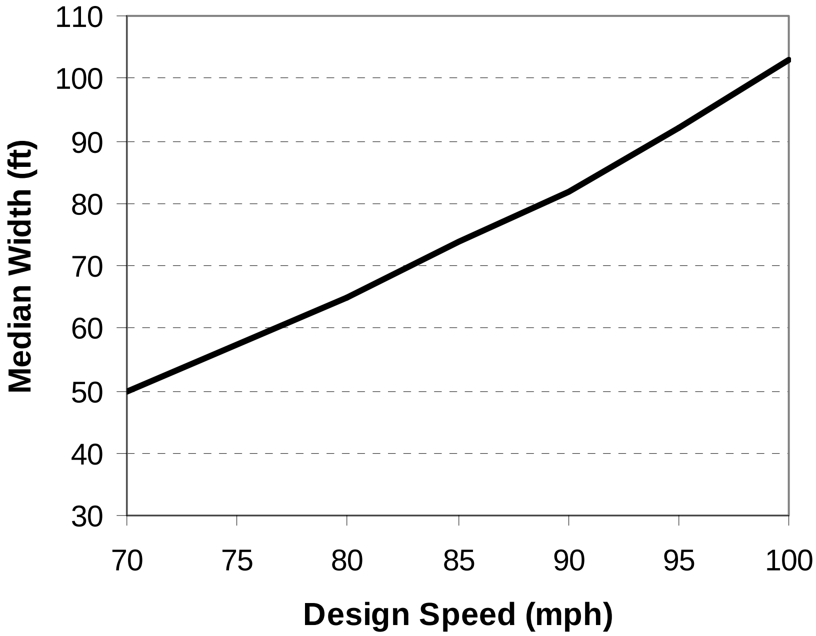
Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 17-7. Extrapolated Median Widths for High Design Speeds.

AASHTO has recently adopted a revised median barrier policy that requires consideration of a barrier for medians having a width of less than 50 ft [15.3 m]. Several states (e.g., California, Arizona, North Carolina) have adopted policies that require median barrier for median widths of 70 ft [21.4 m] or more for design speeds of 70 mph [113 km/h]. Because the relative speed of vehicles at the time of collision is high, cross-median crashes are typically violent and result in multiple injuries and fatalities. For these reasons, it is recommended that the clear zone values derived from braking distance and a median width of 50 ft [15.3 m] at a 70-mph [113 km/h] design speed be adopted for controlled-access facilities with high design speeds.

POTENTIAL MINIMUM MEDIAN WIDTHS FOR HIGH DESIGN SPEEDS

Based on the preceding discussion, and using a 50-ft [15.3 m] median at 70 mph [113 km/h] as a starting point, the potential minimum median widths for high design speeds are shown in Figure 17-8. For high design speeds, the minimum median width ranges from 74 ft [23 m] at 85 mph [137 km/h] to 104 ft [32 m] at 100 mph [160 km/h].



Note: 1 mph = 1.61 km/h, 1 ft = 0.305 m

Figure 17-8. Potential Minimum Median Widths for High Design Speeds.

RESEARCH NEEDS

Because the primary purpose of a median barrier is to prevent an errant vehicle from crossing a median on a divided highway and encountering oncoming traffic, the development of median barrier warrants is often based in part on some B/C analysis of the frequency and severity of cross-median and other median-related crashes. In determining whether or not it is cost-effective to install a barrier, the benefit of reducing the expected frequency and severity of a cross-median crash has to be compared with the cost of installing and maintaining the barrier and generating barrier crashes that would otherwise not have occurred.

Under Project 0-4254, improved guidelines for the use of median barriers on new and existing high-speed, multilane, divided highways in Texas were developed based on a benefit-cost analysis approach. The research approach taken by the study consisted of collecting roadway and median-related crash data to support a "cross-sectional with-without" analysis. The design aimed at estimating and comparing the crash frequency and severity for two groups of sites assembled from a cross section of highways in the same time period: sites with median barriers and sites without median barriers. The highways of interest were those classified as interstates, freeways, and expressways with four or more lanes and have posted speed limits of 55 mph [89 km/h] or higher.

The Poisson-gamma regression models were used to estimate/predict the frequency of cross-median and median-related crashes and an ordered multinomial logit model was used to estimate/predict the severity distributions. These models include variables such as AADT, number of lanes, posted speed limit, median width, etc. The maximum posted speed limit in the data was 70 mph [113 km/h]. The results of this study on Texas median-related crashes could be used to perform model-based extrapolation to extend the models to encompass the higher speeds of interest. In this manner, more detailed guidelines based on Texas median crash data could be established for high-speed facilities.

CHAPTER 18

ROADSIDE SLOPES AND DITCHES

CURRENT GUIDANCE

The TxDOT *Roadway Design Manual* states the following in Chapter 2, Section 6 – Cross Sectional Elements:

> "For safety reasons, it is desirable to design relatively flat areas adjacent to the travelway so that out-of-control vehicles are less likely to turn over, vault, or impact the side of a drainage channel" (*1*).

Design values for the selection of earth fill slope rates in relation to height of fill for different types of terrain are shown in Table 18-1. Ideally, the front slope should be 1V:6H or flatter. Particularly difficult terrain may require deviation from these general values. When the front slope is steeper than 1V:3H, a longitudinal barrier may be needed to keep vehicles from traversing the slope.

Table 18-1. Earth Fill Slope Rates (TRDM, Table 2-10) (*1*).

Height of Fill	Usual Max Slope Rate, Vertical:Horizontal [a]	
	Type of Terrain	
	Flat or Gently Rolling	Rolling
0 – 5 ft [0 – 1.5 m]	1V:8H	1V:6H
5 – 10 ft [1.5 – 3.0 m]	1V:6H	1V:6H
10 – 15 ft [3.0 – 4.5 m]	1V:6H	1V:3H
15 ft and over [4.5 m and over]	Subject to Stability Requirements	
[a] Deviation permitted for particularly difficult terrain conditions.		

Similar guidance can be found in the AASHTO *Green Book* (*2*). In Chapter 4 "Cross Section Elements," it recommends that a rate of slope of 1V:6H or flatter be provided where feasible to provide an errant motorist with a good chance of recovery. It recognizes that site conditions may dictate the use of slopes steeper than desirable. For moderate fill heights with good rounding, slopes up to 1V:3H can also be traversed by vehicles encroaching on the roadside.

OTHER GUIDANCE

The AASHTO *Roadside Design Guide* (*74*) categorizes foreslopes parallel to the flow of traffic as recoverable, non-recoverable, and critical. Recoverable slopes are defined as 1V:4H or flatter. An errant motorist encroaching on a recoverable foreslope can generally regain control and return to the roadway or bring their vehicle to a safe stop.

A non-recoverable slope is defined as "one that is traversable, but from which most vehicles will be unable to stop or return to the roadway easily" (*74*). Slopes between 1V:3H and 1V:4H fall into this category provided they are smooth and free of fixed objects. Because vehicles encroaching onto non-recoverable slopes are expected to reach the bottom of the slope/ditch, the clear zone should typically encompass the entire slope.

Critical foreslopes are defined as those steeper than 1V:3H. Such slopes will cause most encroaching vehicles to overturn and, therefore, should be shielded with a longitudinal barrier if those slopes begin within the clear zone distance of the highway.

DISCUSSION

It is desirable to design relatively flat areas adjacent to the travelway so that out-of-control vehicles are less likely to overturn during a roadside encroachment. However, this is not always practical due to physical and cost restraints at many locations. The question, therefore, becomes "What slopes can be safely traversed by errant vehicles without imparting serious injury to the occupant?" This question is complex to analyze because the trajectory and stability of an errant vehicle that encroaches onto the roadside is a function of a number of factors including, but not limited to, driver input (e.g., steering and/or braking), slope rate and depth, vehicle type, encroachment speed and angle, friction, and soil conditions. Potential means for studying the relationships between these variables include crash data analysis, computer simulation, and full-scale crash testing.

Current guidance on the subject of sideslope rates was developed from research conducted in the late 1960s and early 1970s (*97*, *98*, *99*). Weaver (*97*) used a combination of full-scale tests and computer simulation to develop recommendations for design of ditches for various combinations of side and backslope. A total of 24 full-scale vehicle tests were conducted at three study sites on an unopened four-lane divided highway. The sites had foreslopes ranging from 1V:6.5H to 1V:7.2H, backslopes ranging from 1V:3.3H to 1V:4.9H, and heights ranging from 11 ft [3.4 m] to 17 ft [5.2 m]. A 1963 Ford Galaxy was used for the tests that were conducted at speeds ranging from 30 mph [48 km/h] to 60 mph [97 km/h] and an encroachment angle of 25 degrees. All tests were conducted in a "no-steer" mode. The tests indicated that a 1V:7H sideslope in combination with a 1V:4H or 1V:5H backslope could be safely negotiated at speeds up to 60 mph [97 km/h] with no rollover hazard and only moderate driver discomfort. A 1V:7H sideslope in combination with a 1V:3H backslope was not considered a desirable design.

The results of the full-scale tests, computer simulations, and engineering judgment were used to develop criteria for roadside slope design. The guidelines recommended desirable and absolute maximum backslopes that could be used in combination with sideslopes ranging from 1V:4H to flat based on a design encroachment condition of 60 mph [97 km/h] and 25 degrees.

In another study, Ross and Post and Ross et al. (*98*, *99*) developed criteria for guardrail need on embankments by comparing the severity of impacting a guardrail with the severity of traversing various sideslope and ditch sections. The severity of vehicle encroachments was estimated based on vehicle accelerations obtained from computer simulations. Slopes ranging from 1V:2H to 1V:6H were analyzed in combination with heights ranging from 10 ft [3.1 m] to 50 ft [15.3 m].

A combination of simulations and full-scale crash test data was used to determine the severity associated with impacting a guardrail under selected impact conditions. Guardrail was recommended for situations in which the severity of impacting a guardrail was judged to be less severe than traversing the unprotected embankment. It was concluded that for speeds up to 70 mph [113 km/h] and for encroachment angles less than 17.5 degrees, a collision with a guardrail (at 60 mph [97 km/h] and 25 degrees) is higher in severity than traversing a 3:1 embankment with a 20-ft [6.2 m] fill height.

There are two obvious issues that raise questions about the continued validity of these and similar studies. First and foremost, previous research and current guidance on the subject of slope and ditch traversals are based on passenger car encroachments. Passenger cars are inherently more stable than light trucks (e.g., pickup trucks, sport utility vehicles, vans, and minivans) that now comprise 50 percent of new vehicle sales and constitute a significant percentage of the vehicle fleet. Numerous crash data studies have observed that light trucks are over-represented in rollover crashes compared to passenger cars. Therefore, some roadside slope conditions that are considered traversable for passenger cars may not be traversable for light trucks.

Second, all previous crash tests and most previous simulation studies have involved analysis of vehicle leaving the roadway in a tracking mode (i.e., the rear wheels following the path of the front wheels). Many real-world encroachments occur in a yawing, non-tracking mode (i.e., the velocity vector is not aligned with the vehicle heading angle). Vehicles that leave the roadway in a non-tracking mode have a much higher propensity for overturn than a tracking vehicle. When a vehicle leaves the road in a side slip or spin out, it is more likely to trip and rollover due to wheels furrowing into soft soil or striking an object on the roadside. This result is particularly true for high center-of-gravity light trucks. Therefore, current guidelines tend to underestimate the severity and rollover potential associated with light truck encroachments.

Generally speaking, the percentage of stable tracking vehicle encroachments decreases and the percentage of overturns increases with increasing sideslope ratios. Unfortunately, little data are available regarding the percentage of overturns versus total vehicle encroachments for different sideslope ratios. In a recent analysis of the General Estimates System (GES) database, which is a representative sample of traffic crashes in the U.S. maintained by the National Highway Traffic Safety Administration, it was found that approximately 30 percent of vehicles overturn in crashes in which the object struck is coded as embankment (*100*). However, because the database does not contain information on actual sideslope ratios at the crash sites, more detailed analyses are not possible.

The problem is further complicated when high design speeds are considered. For a given sideslope or ditch section, the probability of rollover is known to be related to vehicle encroachment speed. Additionally, the percentage of non-tracking encroachments on high-speed highways may increase due to drivers' unfamiliarity with vehicle handling and response at high speeds. However, without further study, vehicle behavior during high-speed encroachments cannot be quantified.

POTENTIAL SLOPE AND DITCH VALUES FOR HIGH DESIGN SPEEDS

Lack of data and understanding of driver and vehicle behavior during high-speed encroachments precludes extrapolation of current guidelines for the selection of safe slopes for high-speed roadways. Until further study is performed, engineering judgment is the only option for developing such guidance.

As mentioned above, it is desirable to design slopes as flat as economically feasible so that out-of-control vehicles have better opportunities to recover and are less likely to turn over. Tentative design values for the selection of earth fill slope rates in relation to height of fill are shown in Table 18-2. Particularly difficult terrain may require deviation from these general guide values. Where conditions are favorable, it is desirable to use flatter slopes to enhance roadside safety.

Table 18-2 Tentative Earth Fill Slope Rates for Roadways with High Design Speeds.

Height of Fill	Usual Max[a] Slope Rate, Vertical:Horizontal	
	Type of Terrain	
	Flat or Gently Rolling	Rolling
0 - 5 ft [0 - 1.5 m]	1V:8H	1V:6H
5 ft and over [1.5 m and over]	1V:6H	1V:6H
[a] Deviation permitted for particularly difficult terrain conditions.		

Ideally, the foreslope should be 1V:8H or flatter, although steeper slopes may be acceptable in some locations. The backslope should typically be 1V:6H or flatter. However, the slope ratio of the backslope may vary depending upon the geologic formation encountered. For example, where the roadway alignment traverses through a rock formation area, backslopes are typically much steeper.

The intersections of slope planes in the highway cross section should be well rounded for added safety and increased stability of out-of-control vehicles. Where guardrail is placed on side slopes, the area between the roadway and barrier should be sloped at 1V:10H or flatter.

RESEARCH NEEDS

Research is needed to evaluate the stability of vehicles crossing roadside slopes and ditch configurations at high speeds. The recommended approach is to utilize computer simulation to analyze vehicle trajectory and resulting vehicle kinematics for very high encroachment speeds and combining the results with crash data for purposes of developing encroachment severity/stability relationships for different roadside conditions. The computer simulation matrix could include a wide range of variables known to influence vehicle stability during roadside encroachments. Vehicular resultant accelerations and angular displacements can be captured as output from the simulations and used to help assess encroachment severity, rollover probability, and vehicle stability.

In order to infer the encroachment characteristics of the general vehicle-crash population from discrete simulations, "expansion or weight factors" will be derived from crash data distributions and applied to each encroachment parameter. Because there are no encroachment speed or angle distributions for high-speed roadways, such distributions will need to be extrapolated from crash data on 65- to 70-mph [105 to 113 km/h] speed limit roadways. Under Project 17-22, run-off-road crash cases were selected from the NASS CDS for supplemental field data collection, reconstruction, and clinical analysis. The purpose of the clinical analysis is to develop a database for ran-off-road crashes from which distributions for different characteristics of these crashes (including encroachment speed and angle) can be derived. These distributions can be extrapolated using a parametrical (statistical) regression model.

Using the probabilities assigned to each set of simulated encroachment conditions, the percentage of stable and unstable encroachments can be determined. The data can be further aggregated by speed, sideslope ratio, and vehicle type (passenger cars and light trucks) to examine their relationship on vehicle stability and rollover. For example, one of the outcomes from the analysis could be rollover probability as a function of sideslope ratio.

These results can be used to assess what slopes are recoverable, traversable, and non-traversable at high speeds. This information can then be used to formulate recommended guidance for roadside slopes and ditches for high-speed roadways. The formulation of guidelines will require establishing some criteria or threshold for the percentage of unstable encroachments. One objective means of establishing a suitable threshold is to estimate the percentage of unstable encroachments associated with current slope guidelines from similar simulations for 65- to 70-mph [105 to 113 km/h] roadways, and determine what slope conditions provide this same level of performance at high speeds.

CHAPTER 19

CRASH TESTING

CURRENT GUIDANCE

Guidelines for testing roadside appurtenances originated in 1962 with a one-page document – *Highway Research Circular 482,* "Proposed Full-Scale Testing Procedures for Guardrails" (*101*). This document included four specifications on test article installation, one test vehicle, six test conditions, and three evaluation criteria.

NCHRP *Report 350,* "Recommended Procedures for the Safety Performance Evaluation of Highway Features," published in 1993, is the latest in a series of documents aimed at providing guidance on testing and evaluating roadside safety features (*102*). This 132-page document represented a comprehensive update to crash test and evaluation procedures. It incorporated significant changes and additions to procedures for safety-performance evaluation, and updates reflecting the changing character of the highway network and the vehicles using it. Subsequent to its publication, the Federal Highway Administration adopted *Report 350* as policy through the federal rulemaking process and the document still governs the testing and evaluation of traffic barriers today.

Report 350 uses a 4409-lb [2000 kg] pickup truck as the standard test vehicle to reflect the fact that over one-half of new passenger vehicles sold in the U.S. are in the "light truck" category. This change was made recognizing the differences in wheel bases, bumper heights, body stiffness and structure, front overhang, and other vehicular design factors associated with light trucks. *Report 350* further defines other supplemental test vehicles including a 17,637-lb [8000 kg] single-unit cargo truck and 79,366-lb [36000 kg] tractor-trailer vehicle to provide the basis for optional testing to meet higher performance levels.

Six test levels are defined for longitudinal barriers (e.g., bridge rails, median barriers, guardrails) that place an increasing level of demand on the structural capacity of a barrier system. The basic test level is Test Level 3 (TL-3). The structural adequacy test for this test level consists of a 4409-lb [2000 kg] pickup truck (2000P) impacting a barrier at 62 mph [100 km/h] and 25 degrees. The severity test consists of an 1800-lb [820 kg] passenger car impacting the barrier at 62 mph [100 km/h] and 20 degrees. At a minimum, all barriers on high-speed roadways on the national highway system (NHS) are required to meet TL-3 requirements. Some state departments of transportation require that their bridge railings and/or median barriers meet TL-4, which requires accommodation of a 17,637-lb [8000 kg] single-unit truck impacting a barrier at 50 mph [80 km/h] and 15 degrees. Higher containment barriers are sometimes used when conditions such as a high percentage of truck traffic warrant. Higher test levels (e.g., TL-5 and TL-6) include evaluation with 79,366-lb [36,000 kg] tractor-van trailers and tractor-tank trailers. Such barriers are necessarily taller, stronger, and more expensive to construct.

DISCUSSION

The foreword of NCHRP *Report 350* states the following: "The evolution of the knowledge of roadside safety and performance evaluations is reflected in this document. Inevitably, parts of this document will need to be revised in the future…" (*102*). NCHRP Project 22-14(2), "Improvement of Procedures for the Safety-Performance Evaluation of Roadside Features," was initiated to take the next step in the continued advancement and evolution of roadside safety testing and evaluation (103). Since publication of *Report 350,* changes have occurred in vehicle fleet characteristics and testing technology. The result of Project 22-14(2) will be a new document that will ultimately be published by AASHTO that will supersede NCHRP *Report 350*. A draft document is currently under review by the project panel prior to its submittal to AASHTO through the Technical Committee for Roadside Safety.

At this time, it has been proposed to increase the weight of the pickup truck design test vehicle from 4409 lb [2000 kg] to 5000 lb [2270 kg], change the body style from a ¾-ton [681 kg] standard cab to a ½-ton [454 kg] four-door (i.e., quad-cab), and impose a minimum height for the vertical center of gravity (C.G.) of 28 inches [711 mm]. The increase in vehicle mass represents an increase in impact severity of approximately 14 percent.

Changes to the small car impact conditions have also been proposed. Current recommendations include increasing the weight of the small passenger design test vehicle from 1800 lb [820 kg] to 2425 lb [1100 kg] and increasing the impact angle from 20 degrees to 25 degrees. These changes represent an increase in impact severity of 357 percent.

Passenger Vehicles

The first step in evaluating the performance of current safety hardware or designing new hardware for high-speed roadways is to define the design impact requirements. The design requirements for roadside hardware are historically performance based and described by a test matrix with each test having a prescribed set of impact conditions.

Impact conditions are generally defined by vehicle type, vehicle mass, impact speed, and impact angle. Current guidance on the impact performance criteria for roadside safety features is contained in NCHRP *Report 350*. Six test levels are prescribed for evaluation of longitudinal barriers. The design vehicles range from an 1800-lb [820 kg] passenger car to a 79,637-lb [36,000 kg] tractor-tank trailer combination. The maximum crash test speeds used in NCHRP *Report 350* are nominally 60 mph [97 km/h] for passenger cars and 50 mph [81 km/h] for trucks. It is reasonable to expect that both posted speeds and operating speeds will greatly exceed these values on the high-speed roadways that are the subject of this project.

Determination of impact conditions for single vehicle ran-off-road crashes requires in-depth investigation and reconstruction of detailed crash data. Police-level crash data do not provide sufficient detail for this purpose. Due to the high cost associated with detailed data collection and in-depth crash investigation and reconstruction, few studies of this type have been performed. Two data sources of this nature were developed and analyzed by Mak in the early 1980s. One source included the reconstruction of a statistically representative sample of 472 pole crashes, while the other includes a census of 124 reconstructed crashes involving bridge rails, bridge parapet ends, and approach guardrails.

Using this combined set of in-depth crash data, distributions of impact speed and impact angle were determined for different highway functional classes. Functional class was used as a surrogate measure for the various roadway, roadside, and traffic characteristics that can influence impact speed and angle distributions.

It was found that a gamma function provided the best fit for both univariate impact speed and angle distributions. The data indicated a weak negative correlation between impact speed and impact angle (i.e., higher impact speeds are associated with slightly lower impact angles). However, the researchers were unsuccessful in developing a joint distribution for impact speed and angle and ultimately assumed that the impact speed and angle distributions are independent of one another.

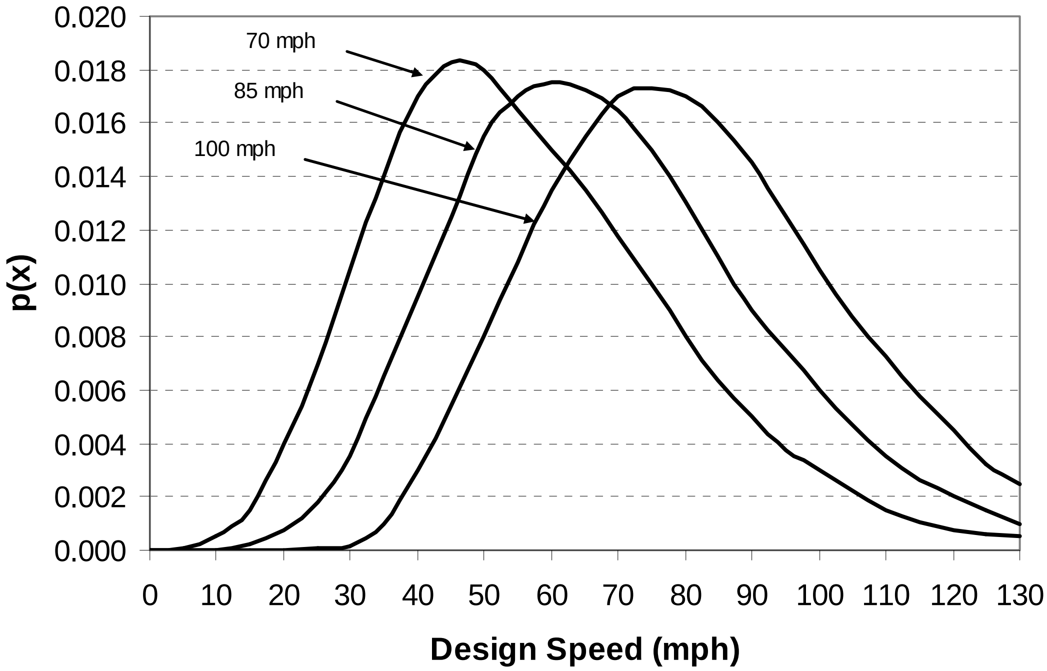
Examination of the impact speed distributions developed by Mak indicates that impact speed varies significantly across functional classes. The current design impact speed of 62 mph [100 km/h] used in the testing and evaluation of roadside safety devices represents the 90th percentile impact speed when all roadway classifications are combined. Additionally, it represents an 85th percentile impact speed for rural arterials, and only a 60th percentile condition for freeways.

Conversely, impact angle was not sensitive to roadway functional class. The current impact angle of 25 degrees used to evaluate the strength of longitudinal barriers represents an 85th percentile impact angle for all roadway classes combined. When only freeways are considered, the 25 degree design impact angle still represents an 85th percentile condition.

When both impact speed and angle are considered, the percentage of real-world crashes that exceed both design conditions is very small. For example, the distributions indicate that only 3 percent of freeway crashes have impact speeds greater than 60 mph [97 km/h] *and* impact angles greater than 25 degrees. That is, only 3 percent of real-world freeway crashes would be expected to exceed the impact severity associated with the combined design speed and design angle. This finding suggests that the current full-scale crash conditions for longitudinal barriers are rather conservative.

If one assumes that the mean impact speed for a given highway functional class is proportional to the design speed of the highway, an extrapolation of the impact speed distribution can be performed. The gamma function is uniquely defined by two coefficients that can be used to describe the mean and variance of the distribution. Therefore, assuming a mean impact speed for a given design speed, a gamma function can be defined.

Figure 19-1 shows the probability density functions for the original freeway data (with an assumed design speed of 70 mph [113 km/h]) and those associated with design speeds of 85 and 100 mph [137 and 160 km/h]. The associated cumulative gamma distribution functions for impact speed are shown in Figure 19-2. Using a 60th percentile as a threshold, the impact speed associated with the selected design speeds can be determined. As summarized in Table 19-1, the design impact speed associated with roadway design speeds of 85 mph [137 km/h] and 100 mph [160 km/h] are 73 mph [118 km/h] and 86 mph [138 km/h], respectively.

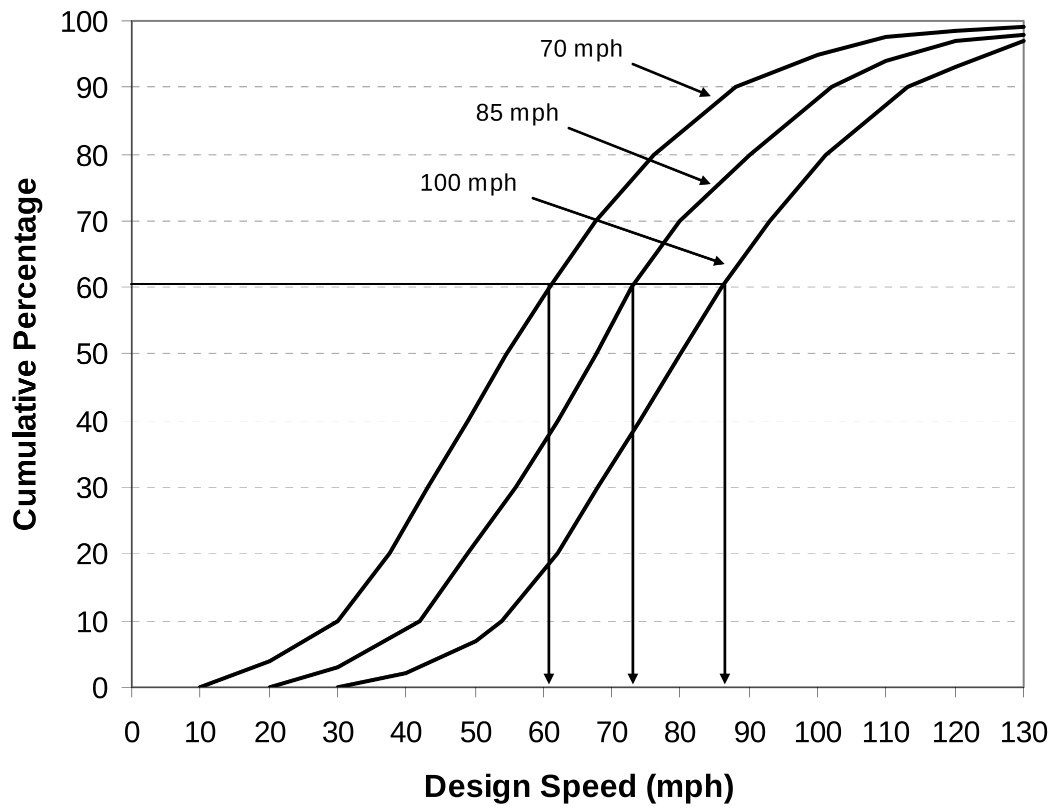


Note: 1 mph = 1.61 km/h

Figure 19-1. Probability Density Functions for Impact Speed as a Function of Design Speed.

Table 19-1. Recommended Impact Speeds for Testing of Roadside Safety Features.

Design Speed mph [km/h]	Impact Speed mph [km/h]
70 [113]	62 [100]
85 [137]	73 [118]
100 [160]	86 [138]



Note: 1 mph = 1.61 km/h

Figure 19-2. Cumulative Distributions of Impact Speed for Different Design Speeds.

As previously discussed, the impact angle distributions derived from the real-world crash data do not vary significantly with functional class. This finding would seem to indicate that impact angle does not vary significantly with design speed. This assumption is supported by the very weak correlation observed between impact speed and impact angle. Therefore, there is little justification for decreasing the impact angle as the impact speed increases. Until better data become available, it is recommended that an impact angle of 25 degrees be maintained for crash testing of roadside safety devices for high speed roadways.

The design vehicles proposed for use in evaluating and/or designing roadside hardware for high-speed roadways will be those recommended in the update of NCHRP *Report 350*. Research is currently being conducted under NCHRP Project 22-14(2) to update *Report 350* and take the next step in the continued advancement and evolution of roadside safety testing and evaluation.

Trucks

Current plans for the Trans-Texas Corridor include separate lanes designated for commercial truck traffic. Other high-speed facilities may also contain a large percentage of truck traffic. There is a need to develop bridge rails, median barriers, and other roadside devices capable of containing and redirecting large, commercial, articulated trucks traveling at high speeds. Such safety devices will be required to provide positive separation between trucks and passenger vehicles, prevent high-speed cross-median crashes between trucks and other vehicles, and to keep trucks contained on bridge structures.

Development of roadside and bridge barriers for high-speed facilities capable of containing trucks requires suitable crash test impact conditions to be defined. Current impact conditions for large trucks recommended in NCHRP *Report 350* involve an 79,637-lb [36,000 kg] tractor-van trailer (TL-5) or tractor-tank trailer (TL-6) impacting a longitudinal barrier at a speed of 50 mph [80 km/h] and an angle of 15 degrees. Devices capable of containing and redirecting trucks under these conditions are applicable for use on highways with a high percentage of truck traffic and design speeds of 70 mph [113 km/h].

Unlike the case with passenger vehicles, real-world impact speed and angle distributions associated with truck crashes are not available for use in extrapolating crash test impact conditions for trucks for highways with high design speeds. In absence of these data, a simple linear extrapolation based on current impact speed and design speed can be used to determine crash test impact speed for trucks for high design speeds. Results of this extrapolation are shown in Table 19-2.

Table 19-2. Recommended Barrier Crash Test Impact Speeds for Trucks.

Design Speed mph [km/h]	Impact Speed mph [km/h]
70 [113]	50 [81]
85 [137]	61 [98]
100 [160]	71 [114]

POTENTIAL IMPACT SPEEDS FOR HIGH DESIGN SPEEDS

The crash test impact speeds for passenger vehicles are summarized in Table 19-1. The impact speeds for trucks are summarized in Table 19-2.

RESEARCH NEEDS

Under NCHRP Project 17-22, "Identification of Vehicular Impact Conditions Associated with Serious Ran-Off-Road Crashes" *(86)*, a robust relational database is being developed for single-vehicle run-off-road crashes. This database will enable researchers to identify the vehicle types, impact conditions, and site characteristics associated with crashes involving roadside features and safety devices. Once available, this database will provide more current and definitive data for assessing the relationships between test impact conditions and actual roadside crashes involving serious injuries and fatalities, and the test impact conditions recommended for high-design-speed facilities.

CHAPTER 20

ROADSIDE SAFETY DEVICES

CURRENT GUIDANCE

For economic reasons, many roadside safety features are optimized for the prescribed design impact conditions and have little or no factor of safety for accommodating more severe impacts. The change in design vehicles proposed for the update to NCHRP *Report 350* will place more structural demand on barrier systems and may aggravate stability problems associated with some existing barriers.

The additional increases in test impact speed recommended for evaluating roadside safety devices for high-speed roadways will unquestionably necessitate the redesign of most roadside appurtenances. For example, the standard metal beam guard fence system used in Texas only marginally contained the 5000-lb pickup truck proposed as the new design vehicle for the update to NCHRP *Report 350* when impacted at a speed of 62 mph [100 km/h]. During redirection, a vertical tear propagated through half the cross section of the W-beam rail. Based on this result, standard guard fence is at its performance limits and will be unable to accommodate the higher test impact speeds recommended for high design speeds without modification. A brief discussion regarding the application of various categories of roadside safety devices on high-design-speed roadways is provided below.

DISCUSSION

Guardrail

As stated in the AASHTO (2002) *Roadside Design Guide*, "A roadside barrier is a longitudinal barrier used to shield motorists from natural or man-made obstacles located along either side of a traveled way" *(74)*. A barrier is typically warranted when the consequences of a vehicle leaving the traveled way and striking a fixed object or traversing a terrain feature are judged to be more severe than striking the barrier. The barrier functions by containing and either capturing or redirecting errant vehicles. The most definitive means of demonstrating the adequacy of the barrier for this purpose is through full-scale crash tests.

In the mid-1990s, TTI researchers conducted full-scale crash tests of all commonly used guardrail systems in accordance with NCHRP *Report 350* Test 3-11 under a pooled fund study administered by FHWA. It was under this testing program that performance issues with the strong steel-post W-beam guardrail (G4[1S]), weak-post W-beam guardrail (G2), and strong steel-post thrie beam guardrail (G9) were first identified. Modifications to these systems were subsequently developed to enable them to accommodate the pickup truck design vehicle.

Under NCHRP Project 22-14(2), the project under which NCHRP *Report 350* is being updated, a limited number of crash tests have been conducted to assess the impact performance of W-beam guardrail when subjected to the revised impact conditions. The standard strong steel-post W-beam guardrail with routed, 8-inch [203 mm] deep wood offset blocks has been shown to be marginal when impacted by a 5000-lb [2270 kg] pickup truck at 62 mph [100 km/h] and 25 degrees. In a test with a ¾-ton [681 kg] standard cab pickup truck ballasted to 5000 lb [2270 kg], the W-beam rail completely ruptured and permitted penetration of the test vehicle. In a subsequent test with a ½-ton [454 kg], four-door, quad-cab pickup truck, the pickup was redirected, but the rail was torn through approximately half of its cross section.

Two modified strong-post W-beam guardrail designs have recently been tested with the proposed heavier pickup truck design vehicle with acceptable results. A system known as the Midwest Guardrail System (MGS) increases the W-beam rail height from 27 inches [686 mm] to 31 inches [787 mm], increases the depth of the offset blocks between the rail and posts from 8 inches [203 mm] to 12 inches [305 mm], and moves the rail splice locations from at a post to midspan between posts. A system known as the T-31 similarly has a rail height of 31 inches [787 mm] and relocates the rail splices to midspan between posts. It also incorporates a proprietary Steel Yielding Line Post (SYLP) that enables the guardrail to function acceptably without offset blocks. It is unknown if either of these systems can accommodate the increased test impact speeds being proposed for high-design-speed roadways.

Median Barrier

As with roadside barriers, median barriers can be generally categorized as weak-post systems, strong-post systems, and continuous concrete barriers. The strong-post W-beam and thrie beam median barriers and weak-post W-beam median barrier incorporate dual-rail elements symmetrically blocked out from the sides of centrally positioned support posts. Due to the inherent severity of crossover crashes, the height of strong-post median barriers is sometimes increased beyond the height-post roadside barriers to provide additional containment capacity. In such designs a rubrail may be used to minimize potential post snagging problems associated with the increased ground clearance of the rail elements. Such strong-post median barrier systems are not presently used in Texas.

As previously discussed, the primary function of a median barrier is to separate opposing traffic and, thereby, prevent severe crossover crashes. Therefore, unlike roadside barriers that commonly shield motorists from discrete hazards (i.e., fixed objects), median barrier is often required along long stretches of highway which makes the low installation cost of weak-post median barriers, such as cable barrier, very appealing. Additionally, the flexibility of these systems results in lower decelerations to an impacting vehicle, which lowers the probability of injury to occupants. However, sufficient space must be available to accommodate the greater design deflections associated with weak-post barrier systems.

High-tension cable barrier systems are rapidly gaining popularity in median applications. The high tension reduces dynamic deflection and enables the cables to remain elevated after an impact. Thus, the barrier retains much of its functionality and can accommodate additional impacts between the time the barrier is impacted and subsequently repaired. The reduced deflection of these systems results in less contact length and, hence, less damage to repair than low-tension cable systems. Thus, repairs can be accomplished at less cost and in shorter time with less risk to maintenance personnel.

The performance of high-tension cable barriers at the test impact speeds proposed for high-design-speed roadways is unknown. However, even if these barriers cannot accommodate the increased impact severity in their current configurations, it is likely that they can be modified to do so. While associated design deflections will almost certainly increase, the expected increases in deflection can be at least partially offset through the use of reduced post spacing.

Concrete barriers are frequently used in narrow medians along high-speed, high-volume roadways due to their negligible deflection, low life-cycle cost, and maintenance-free characteristics. The rigid nature of these concrete barriers results in essentially no dynamic deflection. Thus, vehicle deceleration rates and probability of injury are greater for concrete barriers than for more flexible systems. Although the installation cost is relatively high, concrete barriers require little maintenance or repair after an impact. This reduces the risk of maintenance personnel on high-volume, high-speed roadways.

Concrete median barriers that meet NCHRP *Report 350* requirements include the New Jersey, F-shape, single slope, and vertical wall. While the New Jersey profile has a long history of widespread use, it has been falling out of favor in recent years based on the realization that it can impart significant climb and instability to impacting vehicles. A vertical wall of proper height eliminates issues of vehicle instability, but will impart slightly higher decelerations and cause more damage.

The performance of concrete median barriers at the significantly increased impact speeds proposed for high-design-speed roadways is unknown. There is concern that the increased impact severity will result in unacceptably high acceleration levels and/or, in the case of safety-shape profiles, vehicle instability and rollover.

Bridge Rails

Simply stated, bridge rails are longitudinal barriers that keep vehicles from encroaching off bridge structures and encountering underlying hazards. Bridge rails are typically rigid in nature due to the lack of space on bridge structures to accommodate barrier deflection. Common types of bridge rails include continuous concrete barriers, metal rails mounted on concrete parapets, and both concrete and metal beam and post systems.

Metal rails may be comprised of steel or aluminum material and are typically configured with multiple tubular rail elements mounted to discrete posts bolted to the bridge deck or a continuous concrete parapet. The clear openings between rails and the setback distance of the traffic face of the rails from the support posts must be properly designed to minimize snagging between structural hard points on the vehicle and the support posts. Severe snagging can result in high decelerations, vehicle instability, and/or unacceptable occupant compartment deformation, all of which can increase the probability of injury to occupants of an impacting vehicle.

TxDOT standards include various bridge rails that have been successfully tested or otherwise judged to meet the impact performance requirements of NCHRP *Report 350*. It is uncertain which, if any, of these rails will satisfy the increased impact speeds associated with roadways having high design speeds. As with concrete median barrier, there exist stability concerns for vehicles impacting the T501 safety-shape bridge rail at high speeds. When a New Jersey-profile concrete barrier was crash tested with a pickup truck at 62 mph [100 km/h] and 25 degrees under NCHRP *Report 350*, the barrier imparted significant climb, pitch, and roll to the pickup. A significant increase in impact speed could further aggravate vehicle instability and lead to rollover.

Barrier geometry is not the only source of vehicle instability. Inadequate rail height can also lead to vehicle instability and rollover. Although the 27-inch [686 mm] tall T203 bridge rail meets NCHRP *Report 350* requirements for a TL-3 barrier, it may be unacceptable when impacted at the higher speeds associated with high-design-speed roadways. The increased impact speed will generate greater impact force, increased wheel snagging, and a larger roll moment.

While vehicle stability is not an issue for vertical concrete parapets such as the 32-inch [813 mm] tall T221, it is unknown whether the accelerations imparted by this bridge rail will be acceptable. The accelerations may be above the threshold of serious injury and/or result in unacceptable occupant compartment deformation when impacted by passenger vehicles at high impact speeds.

Crash Cushions

Crash cushions are used to shield motorists from gore areas and other discrete hazards such as bridge piers and overhead sign structures. When impacted head-on, a crash cushion attenuates the energy of a vehicle through various means such as momentum transfer, material deformation, and friction.

The length of a crash cushion is dictated by the amount of energy it must manage in a head-on collision which is a function of the impact speed. Crash cushions designed for high-design-speed roadways will necessarily be longer than the configurations currently approved for Test Level 3 of NCHRP *Report 350*. The stiffness of existing crash cushions will also need to be evaluated to determine if deceleration levels imparted to small passenger cars impacting at high speeds are below thresholds for occupant risk.

Crash cushions must also be able to safely contain and redirect vehicles impacting along their length. In order to redirect vehicles in a stable manner without excessive snagging, pocketing, or occupant compartment deformation, it may be necessary to increase the stiffness of the frame and/or increase the strength of the anchorage system.

Transitions

Transition sections are commonly used to connect a flexible approach guardrail to a more rigid bridge rail. The purpose of the transition is to gradually change the stiffness so that a vehicle impacting the flexible approach rail does not pocket or snag severely on the end of the stiffer bridge rail. The change in stiffness is accomplished through a combination of increased post strength, reduced post spacing, and/or increased rail strength.

As impact speed increases, there is increased potential for pocketing or snagging on the bridge rail end. This, in turn, raises concern regarding vehicle stability and occupant compartment deformation. The current transition design used by TxDOT may require further stiffening to safely contain and redirect passenger vehicles impacting at high speeds. The characteristics of the transition will be dictated by those of the approach guardrail and bridge rails designed to accommodate high design speeds.

End Treatments

Crashworthy end treatments are required to safely terminate guardrail ends. The energy-absorbing end treatments used by TxDOT are designed to dissipate the energy of a vehicle impacting head-on through controlled deformation of the W-beam rail element. Weakened or breakaway posts are used in the end treatment section to help prevent vehicle climbing or vaulting during head-on impacts.

End treatments must also provide anchorage to the guardrail system so that it can function properly to contain and redirect vehicles impacting along its length. The weakened posts in the end treatment section must retain sufficient strong axis strength to resist lateral impact loads generated by an impacting vehicle.

Longer terminal lengths will be required to accommodate end-on impacts at high speed. As impact speeds increase, there is a higher probability that the rail element can buckle, thus hindering the rail extrusion process. Improved anchorage systems may also be required to handle the increased tensile loads associated with containing and redirecting a vehicle impacting at high speed at the beginning of length of need of the terminal.

Breakaway Supports

One of the basic tenets of the “forgiving roadside” concept is to provide an unencumbered, hazard-free recovery area to afford a driver of an errant vehicle a reasonable chance to regain control of the vehicle or bring it to a safe stop. However, it is often necessary to place sign or light support structures in close proximity to edge of travelway to provide information or illumination for motorists. Because these “hazards” serve an important function and cannot be removed, they are designed to break away to minimize the severity of impact.

Breakaway supports can generally be classified into three broad categories: slip base supports, frangible supports, and yielding supports. In a slip base system, two plates are clamped together using three or four slip bolts. Upon impact of the support post, the slip bolts are pushed out of their slots, and the upper plate attached to the support is free to move relative to the fixed lower foundation plate. The vehicle then travels under the rotating support structure.

TxDOT uses slip-base systems for both small sign supports and large guide signs. The performance of slip bases may be satisfactory at high speeds. The increased rotational velocity imparted to the support may be offset by the greater speed of the vehicle. In other words, although the support will be rotating faster, it may not contact the vehicle because the vehicle will be traveling faster beneath it. However, some concern exists regarding the crashworthiness of thin-wall tubing that is commonly used to support small signs with an area of 16 ft^2 [1.5 m^2] or less. At higher impact speeds, the thin-wall tubing may collapse prior to activation of the slip base.

Frangible supports break away by fracturing or failing of components at the base of the support. Cast aluminum transformer bases and frangible anchor studs are examples of frangible breakaway structures. If these devices activate as designed, their performance should be comparable to that of slip-base supports. However, the fracture strength of these materials at very high impact speeds may need to be investigated.

Base-yielding supports typically yield and plastically deform around a vehicle and subsequently experience material failure or pullout from a socket in the ground. Concern exists regarding the crashworthiness of this category of breakaway support when impacted at high speed. If the support does not fracture or release readily enough, it can potentially generate vehicle instability as it wraps around the front of the impacting vehicle or contact the roof and/or windshield of the vehicle and cause unacceptable occupant compartment deformation.

RESEARCH NEEDS

A comprehensive research effort is needed to evaluate the crashworthiness of existing roadside safety hardware when impacted at high speeds. This effort may be accomplished through a combined program of computer simulation and full-scale crash testing.

Recommended impact conditions for evaluation of these devices are presented in the Discussion section above. If analysis or testing identifies deficiencies in a particular device, the device will need to be modified or a new device that meets the intended function will need to be developed.

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