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TRUCK CHARACTERISTICS FOR USE IN HIGHWAY DESIGN AND OPERATION

Volume I, Research Report

U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
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FOREWORD

Many highway design and traffic operational criteria are based in part on vehicle characteristics. Most of these current criteria are based on passenger car characteristics even though truck characteristics may be more critical.

This report, FHWA-RD-89-226, contains information on truck characteristics that are related to the design and operation of highways. The information in the report will be useful to engineers involved in the design of highways with a significant amount of truck traffic. The report will also be useful to persons performing research in the area of highway design.

Sixteen highway design and operational criteria that are based on vehicle characteristics were evaluated in terms of truck operating characteristics. A sensitivity analysis was conducted for each criterion to determine how it varies over a range of truck characteristics. Based on this analysis and considering potential costs and benefits, recommendations are provided that a designer may use to adequately account for trucks in highway design.

Sufficient copies of this report are being distributed to provide a minimum of one copy to each Region and Division office and State highway agency. Direct distribution is being made to the Division offices. Additional copies for the public are available from the National Technical Information Service (NTIS), Department of Commerce, 5285 Port Royal Road, Springfield, Virginia 22161. A small charge will be imposed by NTIS.

R. J. Betsold

R. J. Betsold, Director
Office of Safety and Traffic
Operations Research and Development

NOTICE

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16. Abstract

Highway geometric design and traffic operations are based in part on consideration of vehicle characteristics. However, many of the current highway design and operational criteria are based on passenger car characteristics, even though truck characteristics may be more critical. This report reviews existing data for the truck characteristics that need to be considered in highway design, including truck dimensions, braking distance, driver eye height, acceleration capabilities, speed-maintenance capabilities on grades, turning radius and offtracking characteristics, suspension characteristics, and rollover threshold. The report also includes these truck characteristics. The highway design and operational criteria evaluated include sight distances, vertical curve length, intersection design, critical length of grade, lane width, horizontal curve design, vehicle change intervals at traffic signals, sign placement, and highway capacity. An assessment has been made of the need to change the current highway design and operational criteria to accommodate trucks. The cost effectiveness of proposed changes in design and operational criteria has been evaluated.

Volume II of the report (FHWA-RD-89-227) contains appendixes documenting the detailed data collection and analysis activities.

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SI* (MODERN METRIC) CONVERSION FACTORS

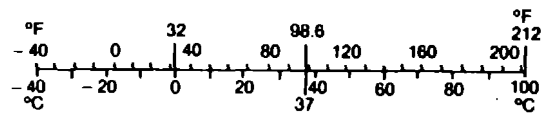
APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
	AREA			
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
	VOLUME			
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3
	MASS			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

NOTE: Volumes greater than 1000 L shall be shown in m^3.

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
	AREA			
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
	VOLUME			
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
	MASS			
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
	TEMPERATURE (exact)			
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F



* SI is the symbol for the International System of Measurement

(Revised April 1989)

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I. INTRODUCTION

Highway design and operational criteria must consider both the characteristics of the vehicles that use the highways and the characteristics of the drivers who operate those vehicles. Driver characteristics for use in highway design, once established, are relatively stable, since human performance characteristics do not change rapidly over time. Vehicle characteristics for use in highway design change continually, however, due both to changes in the dimensions and performance of specific vehicle types and changes in the mix of vehicle types on the road. It is vital that highway design and operational criteria be based on current and future, rather than past, vehicle characteristics.

Many highway design and operational criteria are based either explicitly or implicitly on vehicle characteristics. For example, current passing sight distance requirements are based on an explicit specification of passing vehicle acceleration capability. However, the maneuver distances used in determining passing sight distance contain implicit assumptions about the length of the passing and passed vehicles. It is important that the vehicle characteristics used in design are appropriate for both the current and future vehicle fleet.

There is a critical need to reexamine current highway design and operational criteria to assure that they properly consider vehicle characteristics and, in particular, truck characteristics. The need to focus on truck characteristics arises from the following concerns:

- Trucks are longer, wider, heavier, less maneuverable, and require greater stopping distance than passenger cars or other vehicle types. Thus, trucks are often more critical in highway design and operation than other vehicles.

- Many current highway design and operational standards are based on passenger car characteristics, even though truck characteristics may be more critical.

- Trucks have been increasing as a percentage of the traffic stream. On some Interstate freeways, they constitute 20 to 30 percent of the traffic.

- Trucks have been getting longer, wider, and more powerful. These trends have been accelerated by the 1982 Surface Transportation Assistance Act (STAA) which allowed double-trailer combination trucks and longer, wider, and heavier tractor-semitrailer combination trucks on many roads where they were not previously permitted.

- Engineering research currently under way strongly suggests that future trucks may have more powerful engines, better brakes, and more stable hitches for multiple trailer combinations; yet trucks in the future may be allowed to be still heavier and longer.

Thus, there is a need for a comprehensive review of current highway design and operational criteria to determine whether they are adequate for current and future trucks.

A review of current highway design and operational criteria identified 16 criteria based on vehicle characteristics. These criteria are identified in table 1. Each of these criteria was evaluated in this study.

Table 1. Design and operational criteria based on vehicle characteristics.

- Stopping sight distance
- Passing and no-passing zones on two-lane highways
- Decision sight distance
- Intersection sight distance
- Intersection and channelization geometrics
- Railroad-highway grade crossing sight distance
- Crest vertical curve length
- Sag vertical curve length
- Critical length of grade
- Lane width
- Horizontal curve radius and superelevation
- Pavement widening on horizontal curves
- Cross-slope breaks
- Roadside slopes
- Vehicle change interval
- Sign placement

A. Research Objectives and Scope

The objectives of this research study were to:

1. Identify those highway design and operational criteria that are sensitive to truck performance characteristics.

2. Determine the adequacy of those criteria for trucks.

3. Develop and assess new criteria for those situations where the current criteria do not adequately address the current or future truck population.

The study was primarily analytic in nature. While it was necessary to test and/or measure vehicles to determine certain performance characteristics, such tests were minimized. Whenever possible, existing truck characteristics and truck performance data (such as driver eye height and acceleration capability) were used to determine the sensitivity of various highway design and operational criteria to these characteristics.

The highway design and operational criteria examined in this study included geometric design policies, as well as criteria for signing, signals, and markings. The study scope did not include pavement design criteria (other than pavement surface friction), design of highway structures, or design of roadside hardware.

B. Organization and Scope of This Report

The remainder of the report is organized in three major sections. Section II reviews the truck characteristics that are needed to assess highway design and operational criteria. This section documents the truck characteristics but does not assess their design and operational implications.

Section III of the report assesses the adequacy for trucks of each highway design and operational criterion that is based on a vehicle characteristic. The section documents the current specification for each highway design and operational criterion, presents a critique of that criterion based on the literature, and presents a sensitivity analysis of the effect on that criterion of the differences between current policies (often based on passenger cars or outdated truck data) and the estimates of current (or future) truck characteristics found in section II. Where truck characteristics data were lacking, appropriate assumptions have been made. These assumptions, where critical to determining appropriate highway design and operational criteria for trucks, were documented further and/or validated in special studies presented in the appendixes in volume II of this report.

Although some of the sensitivity analyses in section III of this report imply that current design or operational criteria do not accommodate trucks, these analyses are only one portion of the process for determining appropriate criteria and do not by themselves provide a basis for recommending policy changes. Policy changes are appropriate only if (1) the sensitivity analysis indicates that current design and operational criteria do not accommodate trucks; (2) the policy change would enhance truck safety or operations; (3) the policy change would not degrade safety or operations for other vehicle types, including passenger cars; and (4) the policy change would be cost-effective (i.e., the safety and operational benefits of the policy change would outweigh any increased highway construction costs). These issues have also been addressed in section III of the report.

Section IV of the report presents the conclusions and recommendations of the study.

References cited in the text of the report are listed in section V. Cited references are identified in the text of the report by superscripts. However, two references are cited so often that they are not identified by a reference number each time they are mentioned. These are the American Association of State Highway and Transportation Officials (AASHTO) publication, A Policy on Geometric Design of Highways and Streets - 1984, which is referred to in the text as the AASHTO Green Book, and the Federal Highway Administration (FHWA) Manual on Uniform Traffic Control Devices for Streets and Highways, which is referred to in the text as the MUTCD.[1,2] These publications set the basic criteria currently used in highway design and operation.

Appendix A in volume II of the report presents a detailed discussion of factors influencing truck braking distance, as well as an analysis of new data on truck braking distances collected by NHTSA specifically for this study.

Appendix B in volume II presents an investigation of truck rollovers on horizontal curves using the Phase-4 computer vehicle dynamics simulation model. This appendix addresses the rollover thresholds of specific design trucks and the effect of the type of superelevation transition on the likelihood of truck rollovers on horizontal curves.

Appendix C in volume II addresses the offtracking characteristics of a range of design vehicles, including trucks larger than those addressed in the 1984 AASHTO Green Book. The appendix includes a new model of truck offtracking on horizontal curves that addresses the contributions to offtracking of vehicle speed and pavement superelevation.

Appendix D in volume II addresses recent trends in truck performance on grades and includes a reanalysis of existing truck performance data to derive appropriate truck weight-to-power ratios for use in climbing lane warrants.

Appendix E in volume II reports the results of pilot field studies to establish data collection techniques for evaluation of intersection sight distance requirements for trucks. The results of the field studies include some preliminary estimates of gap acceptance, deceleration rates, acceleration rates, and minimum separations for use in deriving intersection sight distance criteria for trucks.

Appendix F in volume II documents the methodology for cost-effectiveness analyses of candidate revisions to highway design and operational criteria for trucks used in the study. The cost-effectiveness methodology is illustrated by several examples related to revised stopping sight distance criteria for trucks.

The data presented in all of the appendixes in volume II are used in appropriate places in this volume to determine truck characteristics for use in sensitivity analyses and to determine the cost-effectiveness of candidate changes in highway design and operational criteria.

II. TRUCK CHARACTERISTICS

This section of the report reviews the available data on truck characteristics that need to be considered in the development of highway design and operational criteria. The review of truck characteristics is based primarily on data from existing sources in published and unpublished literature. Gaps in the existing state of knowledge are noted.

The review focuses primarily on the characteristics of the current truck population. The effects of recent trends in trucking and recent legislative changes, such as the 1982 Surface Transportation Assistance Act (STAA), are accounted for whenever possible. Where current trends in truck characteristics are evident and truck characteristics may be changed in the near-term, projected future truck characteristics are also addressed. For example, the review recommends that highway design criteria should consider the effects of a tractor-semitrailer design vehicle with a 53-ft (16.2-m) semitrailer length that is likely to become more common in the future; the improvement in truck braking distances that can be expected if antilock brake systems come into widespread use should also be considered.

This review of truck characteristics provides the basic data used in section III to consider the highway design and operational criteria that would be suitable for trucks. Thus, the review is selective, rather than exhaustive; it focuses on the data needed for the analyses in section III. For example, some frequently discussed truck safety issues, such as rearward amplification in emergency steering maneuvers by multitrailer combinations, are not discussed because they have no clear implications for highway design and operational criteria. Many such truck safety issues are more in the realm of truck policy and vehicle design than geometric design or traffic operations.

More complete reviews of many specific truck characteristics can be found in the references cited. In particular, the National Highway Traffic Safety Administration (NHTSA) report, "Heavy Truck Safety Study," provides an excellent overview of many truck design issues and another NHTSA report, "A Factbook of the Mechanical Properties of the Components for Single-Unit and Articulated Heavy Trucks," provides the most detailed available data on the ranges of specific truck characteristics.[3,4]

A. Truck Dimensions

1. Current Design Vehicles

The AASHTO Green Book includes ten design vehicles for use in highway design: a passenger car, four trucks, two buses, and three recreational vehicles. The design vehicles are used as the basis for design criteria for which vehicle length, width, height, or offtracking are controlling factors. Table 2, which reproduces table II-1 in the AASHTO Green Book, shows the

Table 2. AASHTO design vehicle dimensions.[1]

Design Vehicle Type	Symbol	Dimension (ft) Overall Height	Width	Length	Overhang Front	Rear	WB_1	WB_2	S	T	WB_3
Passenger car	P	4.25	7	19	3	5	11				
Single unit truck	SU	13.5	8.5	30	4	6	20				
Single unit bus	BUS	13.5	8.5	40	7	8	25				
Articulated bus	A-BUS	10.5	8.5	60	8.5	9.5	18		4[a]	20[a]	
Combination trucks											
Intermediate semitrailer	WB-40	13.5	8.5	50	4	6	13	27			
Large semitrailer	WB-50	13.5	8.5	55	3	2	20	30			
"Double Bottom" semi-trailer – full-trailer	WB-60	13.5	8.5	65	2	3	9.7	20	4[b]	5.4[b]	20.9
Recreation vehicles											
Motor home	MH		8	30	4	6	20				
Car and camper trailer	P/T		8	49	3	10	11	5	19		
Car and boat trailer	P/B		8	42	3	8	11	5	15		

a = Combined dimension 24, split is estimated.
b = Combined dimension 9, 4, split is estimated
WB_1, WB_2, WB_3, are effective vehicle wheelbases.
S is the distance from the rear effective axle to the hitch point.
T is the distance from the hitch point to the lead effective axle of the following unit.
NOTE: 1 ft = 0.305 m

dimensions for the ten design vehicles. AASHTO specifies that the design of highway facilities should be based on the largest (or least maneuverable) design vehicle likely to use the facility with considerable frequency or on a design vehicle with special characteristics appropriate for that facility. The following discussion considers the changes needed for this set of design vehicles, particularly in light of the 1982 STAA.

The four design trucks specified by AASHTO include a single-unit truck (SU), an intermediate semitrailer truck (WB-40), a large semitrailer truck (WB-50), and a "double bottom" semitrailer-full trailer truck (WB-60). The SU, WB-40, and WB-50 design vehicles are unchanged from the 1965 AASHO Blue Book.[4] The WB-60 design vehicle was first added in the 1984 Green Book.

It should be noted that, although the WB-50 may have been considered a large semitrailer truck at the time of the 1965 Blue Book, it would no longer have been considered a large semitrailer even before the 1982 STAA. The WB-50 has a semitrailer about 38 ft (11.6 m) long. The most common semitrailer lengths in the truck fleet prior to the 1982 STAA were 40 and 45 ft (12.2- and 13.7-m). In updating the current design criteria, it would be desirable to replace the WB-50 with a typical truck with a 40- or 45-ft (12.2- and 13.7-m) semitrailer. This design vehicle would be appropriate for use at many locations off of the designated highway system created by the 1982 STAA.

By contrast, the WB-60 design vehicle does represent the most common twin trailer truck in use prior to the 1982 STAA.

The combination trucks currently used as design vehicles all have heights of 13.5 ft (4.1 m) and widths of 8.5 ft (2.6 m). The STAA increased the allowable width for many trucks from 8 ft (2.4 m) to 8.5 ft (2.6 m). In this case, truck regulations have now caught up with highway design policies. Further changes in truck height and truck width are not expected in the near future.

2. Recommended Design Vehicles

There is a need to update current design criteria, especially those related to vehicle length and offtracking, for the larger trucks permitted by the 1982 STAA on the designated highway system. Four new design vehicles are recommended. These are:

- STAA single with 48-ft (14.6-m) semitrailer.
- Long single with 53-ft (16.2-m) semitrailer.
- STAA double with two 28-ft (8.5-m) trailers and a cab-over-engine tractor.
- STAA double with two 28-ft (8.5-m) trailers and a conventional cab-behind-engine tractor.

The key dimensions for these recommended design vehicles are shown in table 3, using the same dimensional elements as in the AASHTO Green Book. Table 4 gives more complete data on the spacings between the axles and hitch points of the recommended design vehicles. Dimension D in table 4 is applicable to semitrailers with the rear axles positioned as close to the rear of the truck as possible. The rear axles of long semitrailers can often be moved forward from that position to reduce the kingpin-to-rear-axle distance. The implications of this practice for truck operations and safety are discussed in section III-E.

The STAA single with a 48-ft (14.6-m) semitrailer is permitted by the STAA to run anywhere on the national network for trucks. This design vehicle is the most appropriate to represent the current fleet of tractor-semitrailers. The dimensions shown for this design vehicle in tables 3 and 4, including the overall length of 63.5 to 65.5 ft (19.4 to 20.0 m), are based on unpublished data from a recent study and published data from the 1989 Transportation Research Board (TRB) study of truck access requirements.[5,6] In contrast, another recent TRB study found a shorter (60-ft or 18.3-m) overall length for a truck with 48-ft (14.6-m) semitrailer and the California Department of Transportation highway design manual uses a sightly longer (68-ft or 20.7-m) truck.[7,8] Such variations are to be expected, since the 1982 STAA limits trailer lengths rather than overall truck lengths.

The 1982 STAA included "grandfathering" provisions that require States that already allowed semitrailers longer than 48 ft (14.6 m) in length to continue to allow them, at least on the designated highway system. Thirty-four (34) States currently allow 53-ft (16.2-m) semitrailers on at least some facilities, either under the STAA "grandfathering" provisions, under permit, or on specific toll roads. However, data from the Truck Trailer Manufacturers Association indicate that only about 5.8 percent of the trailers that were

Table 3. Recommended dimensions for longer design vehicles.

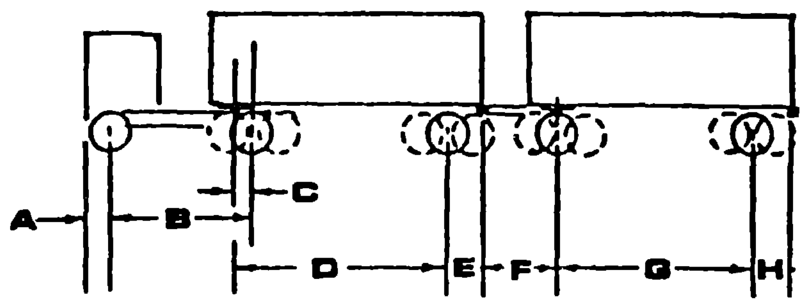
Design vehicle	Dimension (ft)									
	Overall			Overhang						
	Height	Width	Length	Front	Rear	WB_1	WB_2	S	T	WB_3
STAA single with 48-ft trailer	13.5	8.5	63.5-65.5	2.5	4.5	18.0	38.0-40.0	40.0	-	-
Long single with 53-ft trailer	13.5	8.5	68.5-70.5	2.5	4.5	18.0	43.0-45.0	45.0	-	-
STAA double with cab-over-engine tractor	13.5	8.5	66.5-68.5	2.5	2.5	10.0	20.5-22.5	22.5	6.0	22.5
STAA double with conventional tractor	13.5	8.5	69.5-71.5	2.5	2.5	13.0	20.5-22.5	22.5	6.0	22.5

Note: WB, WB_1, WB_2, S, T, and WB_3 are defined in table 2.
1 ft = 0.305 m

Table 4. Detailed axle spacings for longer design vehicles.

Design vehicle	Dimension (ft) A	B	C	D	E	F	G	H	Overall length
STAA single with 48-ft trailer	2.5	18.0	0.0-2.0	40.5	4.5	-	-	-	63.5-65.5
Long single with 53-ft trailer	2.5	18.0	0.0-2.0	45.5	4.5	-	-	-	68.5-70.5
STAA double with cab-over-engine tractor	2.5	10.0	0.0-2.0	22.5	2.5	6.0	22.5	2.5	66.5-68.5
STAA double with cab-behind-engine tractor	2.5	13.0	0.0-2.0	22.5	2.5	6.0	22.5	2.5	69.5-71.5

Note: Dimensions A through H are defined below.
1 ft = 0.305 m



manufactured in 1988 have lengths over 48 ft (14.6 m); the comparable value in 1986 was 2.5 percent, so use of 53-ft (16.2-m) trailers is increasing.[9] The long single with a 53-ft (16.2-m) semitrailer may not currently be used widely enough to constitute a design vehicle for nationwide application. However, it is appropriate for current use on roads where 53-ft (16.2-m) semitrailers are allowed and it may become appropriate for nationwide use in the future.

The most common STAA double has two 28-ft (8.5-m) trailers and a two-axle cab-over-engine tractor. The dimensions of this design vehicle shown in tables 3 and 4 are based on estimates from two recent studies which recommend nearly the same dimensions.[5,7] This design vehicle has an overall length of 66.5 to 68.5 ft (20.3 to 20.9 m) depending on the fifth wheel offset.

The TRB study indicates that most twin trailer operators use two-axle tractors and intend to continue to do so.[7] However, with no overall truck length limits under the STAA, there is nothing that requires twin trailer truck operators to continue to use the cab-over-engine tractor. The TRB study indicates that twin trailer operators were uncertain about their plans to move to conventional cab-behind-engine tractors. The use of conventional tractors for twin trailer trucks appears likely to grow, so a STAA double-trailer design vehicle with a longer tractor has also been provided. The trailers for this design vehicle are identical to the previous design vehicle, but the tractor wheelbase has been increased to 13 ft (4.0 m), which is still shorter than the maximum wheelbase of 17 ft (5.2 m) for a two-axle tractor found from NHTSA data.[4] The STAA double with a conventional tractor is the longest of the STAA design vehicles, with an overall length of 69.5 to 71.5 ft (21.2 to 21.8 m). Several sensitivity analyses in section III of this report use a maximum truck length of 75 ft (22.9 m), which is a conservative choice, slightly larger than the largest recommended design vehicle.

3. Longer Combination Vehicles

Longer combination vehicles (LCVs), including Rocky Mountain doubles, turnpike doubles, and triples, are permitted to operate in a few States. Such trucks can have lengths up to 115 ft (35.1 m). The specific dimensions of these vehicles have been tabulated in the FHWA LCV study and another recent study.[5,10,11] These dimensions are of interest in the specific States that permit their use, but LCVs are not appropriate as design vehicles for general use at this time.

B. Braking Distance

Braking distance is defined in the AASHTO Green Book as "the distance required to stop the vehicle from the instant brake application begins." Braking distance is used in the determination of many highway design and operational criteria, including stopping sight distance, intersection sight distance, vehicle change intervals for traffic signals, and advance warning sign placement distances. Currently, all of these design and operational criteria are based on passenger car braking distances and do not consider the longer braking distances required for trucks. The process of bringing a truck to a stop requires a complex interaction between the driver, the brake system, the truck tires, the dimensions and loading characteristics of the truck, and the pavement surface characteristics. Because truck braking is much more complex than passenger car braking, it is necessary to discuss the role of each of these characteristics in truck braking distances.

1. Tire-Pavement Friction in Braking Maneuvers

Vehicles are brought to a stop by brakes that retard the rotation of the wheels and allow tire-pavement friction forces to decelerate the vehicle. An understanding of the forces involved in tire-pavement friction is, therefore, critical to the understanding of braking distances.

The coefficient of braking friction (f_y) is defined as the ratio of the braking force (F_y) generated at the tire-pavement interface to the vertical load (F_z) carried by the tire. In other words:

$$f_y = \frac{F_y}{F_z} \tag{1}$$

On a horizontal curve, tire-pavement friction also supplies a cornering force to keep the vehicle from skidding off the road. The coefficient of cornering friction (f_x) is the ratio of the cornering force (F_x) generated at the tire-pavement interface to the vertical load (F_z) carried by the tire. In other words:

$$f_x = \frac{F_x}{F_z} \tag{2}$$

Figure 1 illustrates that both braking and cornering friction vary as a function of percent slip, which is the percent decrease in the angular velocity of a wheel relative to the pavement surface as a vehicle undergoes braking. A freely rolling wheel is operating at zero percent slip. A locked wheel is operating at 100 percent slip with the tire sliding across the pavement. Figure 1 shows that the coefficient of braking friction increases rapidly with percent slip to a peak value that typically occurs between 10 and 15 percent slip. The coefficient of braking friction then decreases as percent slip increases, reaching a level known as the coefficient of sliding friction at 100 percent slip.

The coefficient of cornering friction has its maximum value at zero percent slip and decreases to a minimum at 100 percent slip. Thus, when a braking vehicle locks its wheels, it may lose its steering capability due to a lack of cornering friction.

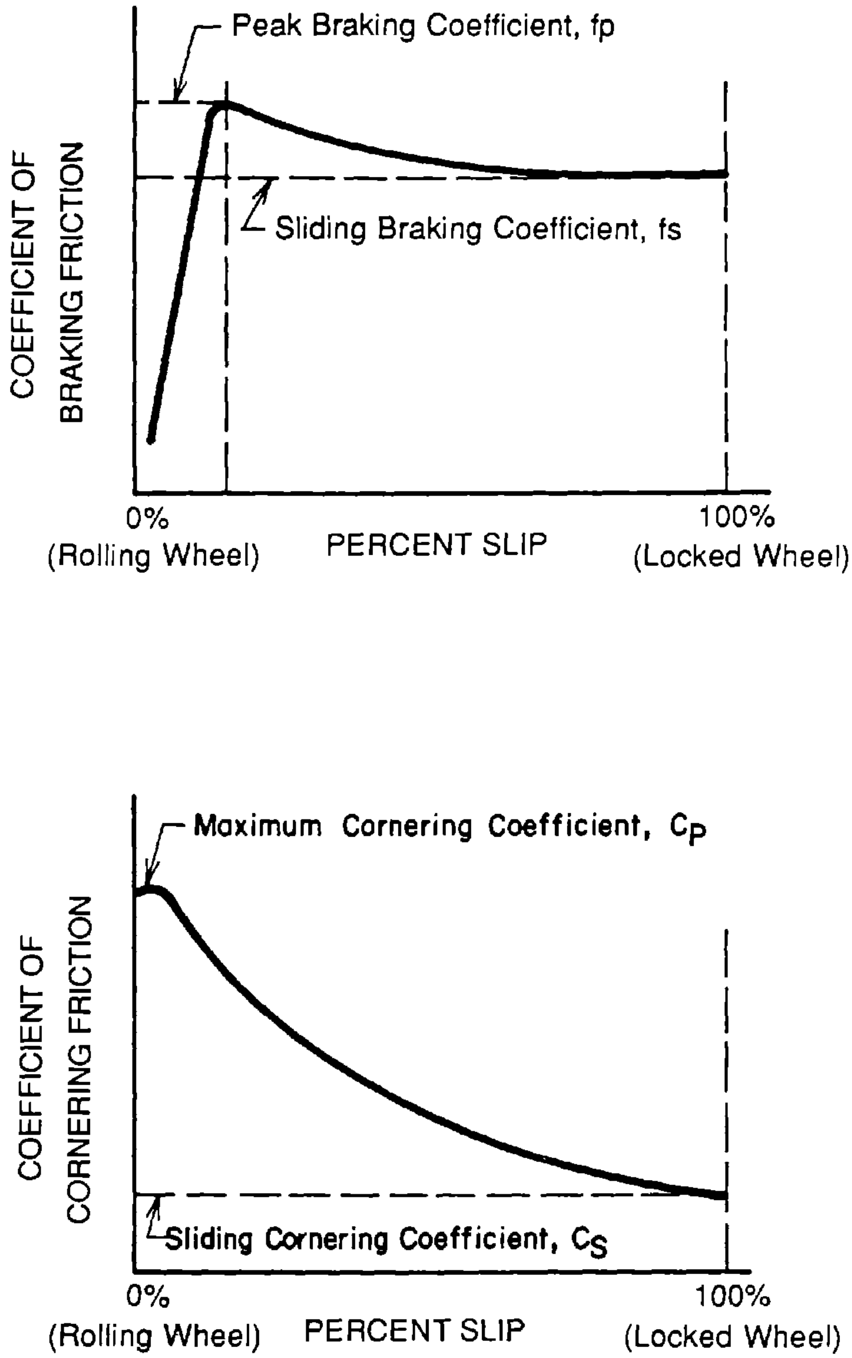


Figure 1. Variation of braking and cornering friction coefficients with percent slip.

2. Locked Wheel Braking vs. Controlled Braking

The discussion of figure 1 implies that braking maneuvers can be performed in two general modes: locked wheel braking and controlled braking. Locked wheel braking occurs when the brakes grip the wheels tightly enough to cause them to stop rotating, or "lock," before the vehicle has come to a stop. Braking in this mode causes the vehicle to slide or skid over the pavement surface on its tires. Locked wheel braking uses sliding friction (100 percent slip) represented by the right end of the graph in figure 1, rather than rolling or peak friction. The sliding coefficient of friction takes advantage of most of the friction available from the pavement surface, but is generally less than the peak available friction. On dry pavements, the peak coefficient of friction is relatively high with very little decrease in friction at 100 percent slip. On wet pavements, the peak friction is lower and the decrease in friction at 100 percent slip is generally larger.

The braking distance required for a vehicle to make a locked wheel stop can be determined from the following relationship:

$$BD = \frac{V^2}{30f_s} \tag{3}$$

where: BD = Braking distance (ft)

V = Initial speed (mi/h)

f_s = Coefficient of sliding friction

The coefficient of sliding friction in equation (3) is mathematically equivalent to the deceleration rate used by the vehicle expressed as a fraction of the acceleration of gravity (g), equal to 32.2 ft/sec² (9.8 m/s²). The coefficient of friction and, thus, the deceleration rate may vary as a function of speed during the stop, so f_s in equation (3) should be understood as the average coefficient of friction or deceleration rate during the stop.

Controlled braking is the application of the brakes in such a way that the wheels continue to roll without locking up while the vehicle is decelerating. Drivers generally achieve controlled braking by "modulating" the brake pedal to vary the braking force and to avoid locking the wheels. Controlled braking distances are governed by the rolling coefficient of friction, which, for a typical truck, occurs at a value of percent slip to the left of the peak available friction shown in figure 1. Due to the steep slope of the braking friction curve to the left of the peak and due to braking techniques used by drivers to avoid wheel lock up, the average rolling friction utilized by trucks is generally less than the sliding friction coefficient. Therefore, controlled braking distances are usually longer than locked wheel braking distances, although theoretically they would be less if the driver could use peak braking friction.

Locked wheel braking is commonly used by passenger car drivers during emergency situations. Passenger cars can often stop in a stable manner, even with the front wheels locked. In this situation the driver loses steering control, and the vehicle generally slides straight ahead. On a tangent section of road this is perhaps acceptable behavior, although on a horizontal curve the vehicle may leave its lane, and possibly the roadway.

Trucks, by contrast, have much more difficulty stopping in the locked-wheel mode. Figure 2 illustrates the dynamics of a tractor-trailer truck if its wheels are locked during emergency braking.[3] The behavior depends upon which axle locks first--they usually do not all lock up together. When the steering wheels (front axle) are locked, steering control is eliminated, but the truck maintains rotational stability and it will skid straight ahead. However, if the rear wheels of the tractor are locked, that axle(s) slides and the tractor rotates or spins, resulting in a "jackknife" loss of control. If the trailer wheels are locked, those axles will slide and the trailer will rotate out from behind the tractor which also leads to loss of control. Although a skilled driver can recover from the trailer swing through quick reaction, the jackknife situation is not correctable. None of these locked-wheel stopping scenarios for trucks are considered safe. Therefore, it is essential that trucks stop in a controlled braking mode and that highway design and operational criteria recognize the longer distances required for trucks to make a controlled stop.

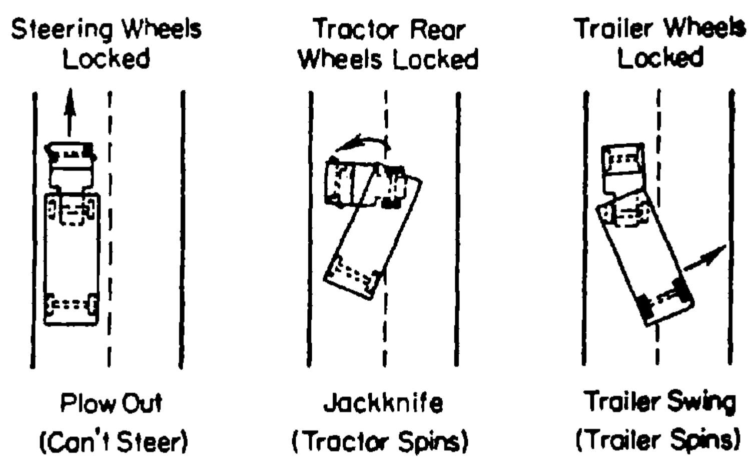


Figure 2. Tractor-trailer dynamics with locked wheels.[3]

The braking distance for a vehicle to make a controlled stop can be determined from the following relationship:

$$BD = \frac{V^2}{30f_r} \qquad (4)$$

where: BD = Braking distance (ft)
f_r = Coefficient of rolling friction
V = Initial speed (mi/h)

As in the case of sliding friction, the coefficient of rolling friction (f_r) in equation (4) represents the average coefficient of friction or average deceleration rate during the entire controlled stop.

3. Pavement and Truck Characteristics Affecting Braking Distance

In order to stop without the risk of loss of control, trucks must use controlled braking rather than locked wheel braking. The deceleration rates used by trucks in making a controlled stop are represented by f_r in equation (4). The following discussion reviews the individual pavement and tire characteristics that affect the value of f_r and, thus, the braking distance of a truck. Appendix A discusses the role of additional factors that affect braking distance including road roughness, brake adjustment, and brake lining temperature.

a. Pavement Properties

The shape of the braking friction curve in figure 1 is a function of both pavement and tire properties. Highway agencies generally measure pavement friction by means of locked-wheel skid tests with a standard tire. These tests determine a value equivalent to f_s in equation (3). The results of these tests are often multiplied by 100 and referred to as skid numbers rather than coefficients of friction. Although skid numbers are usually determined at 40 mi/h (64 km/h), a procedure is available to determine the skid number at any speed from the skid number at 40 mi/h (64 km/h).[12,13,14] The peak coefficient of friction (f_p) can be estimated from the sliding coefficient of friction by the following relationship:[12]

$$f_p = 1.45 f_s \qquad (5)$$

Equation (5) represents the average relationship for truck tires between peak and sliding friction; this relationship can vary markedly between pavements and for the same pavement under wet and dry conditions. Pavements generally have much lower coefficients of friction under wet conditions than under dry conditions, so highway design criteria are generally based on wet conditions.

Estimates of braking distance in a recent evaluation of stopping sight distance requirements in NCHRP Report 270 used an assumed pavement skid number at 40 mi/h (64 km/h) (SN_{40}) of 28.[12] The AASHTO Green Book criteria for stopping sight distance are based on a pavement with SN_{40} equal to 32.

b. Tire properties

Truck tires are designed primarily for wear resistance. For this reason, they tend to have somewhat lower wet friction coefficients than passenger car tires. It is generally estimated that truck tires have coefficients of friction that are about 70 percent of those of passenger car tires.[12] However, passenger car tires generally have coefficients of friction that are about 120 percent of the friction coefficients of the standard tires used in skid testing. Thus, the peak coefficient of friction can be estimated from skid test results with the following relationship:

$$f_p = (1.20)(0.70)(1.45)\ f_s = 0.0122\ SN_{40} \tag{6}$$

The coefficient of friction for truck tires decreases as the tires wear and their tread depth decreases. New truck tires have tread depths of 15/32 in (1.2 cm) for ribbed tires and 31/32 in (2.5 cm) for lug type tires. NCHRP Report 270 assumes, based on the literature, that the tread wear of truck tires has very little effect on their frictional properties until the tread depth falls below 12/32 in (1.0 cm).[12,15] Tire tread depth has little effect on the coefficient of friction on pavements with high macrotexture, but that the coefficient of friction does decrease substantially with tread depth for smooth, poorly textured pavements.[16] The following relationship was used in NCHRP Report 270 to estimate the reduction in friction coefficient of tires as their tread depth decreases:[12]

$$TF = 1 - \frac{\Delta f_p \left(1 - \sqrt{x/n}\right)}{f_p} \tag{7}$$

where, TF = adjustment factor for tire tread depth

Δf_p = difference in coefficient of friction between new and bald (completely worn) tires

x = remaining tread depth (in) (use 12/32 if $x \geq 12/32$)

n = minimum tread depth with coefficient of friction equivalent to a new tire (assumed: 12/32 in or 1.0 cm)

Equation (7) is apparently based on studies of passenger car tires, but no equivalent relationship for truck tires is currently available.

Data on the coefficients of friction for various types of truck tires are available in references 4, 16, 17, and 18. Both references 16 and 17 indicate that the friction coefficients of truck tires decrease slightly with increasing axle load. Tire inflation pressure has very little effect on peak friction coefficient (f_p), but increasing the inflation pressure from 68 to

102 psi (47 to 70 kPa) results in a very small loss (less than 10 percent) in the sliding friction coefficient (f_s).[18]

c. Braking Efficiency

Current truck braking systems are limited in their ability to take advantage of all of the friction available at the tire-pavement interface. Fancher has estimated that the braking efficiency for single-unit trucks is between 55 and 59 percent of the peak available friction.[19] Both Fancher and NCHRP Report 270 assume that this same level of braking efficiency is applicable to tractor-trailer trucks.[12,19] A primary reason for this relatively low level of braking efficiency is that most controlled braking takes place at a value of percent slip less than level which produces the peak braking friction coefficient. Several other vehicle-related factors that contribute to low braking efficiencies are reviewed in this section. Factors, such as antilock brake systems, that might lead to future increases in braking efficiency are also discussed.

By way of introduction, the operation of air brakes--the most common braking system for trucks--is reviewed. Air brake systems use compressed air to transmit and amplify the driver's input from the brake pedal to the brakes on individual wheels. The use of air as an amplifying medium results in a slight delay in the system response due to the compressibility of air. (In contrast, hydraulic braking systems provide an almost immediate response). Once the brake pedal is released, the air in the system is expelled to the atmosphere and is replaced by air from a compressor on board the truck. Therefore, air brakes are not "pumped," as might be done in making a controlled stop with hydraulic brakes. Pumping of air brakes will result in the rapid depletion of the compressed air supply which in turn results in a total loss of braking ability. Rather, for an air brake system, the pressure within the system is adjusted by slightly depressing or slightly releasing the brake pedal to apply more or less braking force. This braking practice is called "modulating" the brakes. As discussed earlier in this section, "modulating" the brakes requires some experience on the part of the driver to obtain the maximum braking effect from the system without causing the wheels to lock.

Loading configuration: Braking tests for tractor-trailer combinations have generally found that loaded trucks have the shortest (controlled) braking distances. Empty trucks generally have longer braking distances and bobtail tractors (with no trailer attached) have the longest braking distances. Some comparative braking distances for these loading configurations are presented in the review of braking test results later in this section. These differences occur primarily because the truck braking system is designed to be balanced for the loaded condition and is, therefore, out of balance for the empty and bobtail conditions.

Technology improvements to braking systems may minimize the effects of loading conditions in future years. For example, some tractors are already equipped with a sensor in the "gladhand" brake line connection that detects whether or not a trailer is attached and adjusts the brakes on the drive axle of the tractor accordingly. Future trucks may have microprocessor controlled braking systems with load sensors on each axle to adjust the braking system accordingly. At present, conservative estimates of braking distance should be based on an empty tractor-trailer truck.

Disconnection of front-axle brakes: For many years, truckers in the United States have disconnected the front-axle brakes of their trucks. Although this practice is now illegal, it became widespread because of concern that the driver might lose control of the truck if the front-axle brakes were locked in an emergency situation. Figure 2 illustrates that while locked front-axle brakes may lead to the inability to steer, this is potentially much less hazardous than locking the brakes on other axles of the truck. Tests by NHTSA have shown that trucks with disconnected front brakes require 20 to 25 percent greater braking distance.[20] Enforcement activities to assure that front brakes are not disconnected have been increased.

Automatic limiting valves for front-axle brakes: A new component added to braking systems that has gained popularity in recent years is an automatic limiting valve for the front-axle brakes. The purpose of the automatic limiting valve is to limit the amount of braking achievable on the front axle. According to NHTSA, approximately two-thirds of post-1980 combination unit trucks are equipped with automatic limiting valves.[3] The advantage of an automatic limiting valve is that it reduces the possibility of wheel lock on the steering axle, which means the driver retains steering control during heavy application of the brakes, even if other wheels might lock. The main disadvantage is that, similar to disconnection of the front-axle brakes, an automatic limiting valve reduces the braking capability of the truck, which lengthens the braking distance. Table 5 presents data for controlled stops by trucks with and without automatic limiting valves.[3]

Table 5. Braking distances for trucks with and without automatic limiting valves for front-axle brakes.[3]

60 mi/h, empty, straight line stop	
Single-unit truck with three axles	440 to 355 ft
Bobtail tractor with three axles	418 to 324 ft
50 mi/h, empty, 500 ft radius curve, wet asphalt	
Single-unit truck with three axles	268 to 233 ft
Tractor-semitrailer (2S1)	260 to 224 ft
Bobtail tractor with two axles	308 to 249 ft
Auto transport truck (stinger)	215 to 181 ft
18 mi/h, loaded, 500 ft radius curve, ice	
Tractor-semitrailer (3S2)	273 to 253 ft
Tractor-semitrailer (2S1)	213 to 179 ft

Note: 1 mi = 1.61 km
1 ft = 0.305 m

In all cases, the shorter braking distance in each range shown in table 5 is the braking distance without an automatic limiting valve. The increase in braking distance resulting from use of an automatic limiting valve ranges from 8 to 29 percent.

Antilock brake systems: During the mid 1970's, regulations for truck braking distances were adopted, which resulted in the introduction of antilock brake systems on trucks. Shortly afterwards, the restrictions were removed by court order and, due to a lack of consumer interest, trucks equipped with antilock brakes were no longer commercially available from domestic truck manufacturers. Since that time, with technological advancements and improved design, antilock braking systems have gained acceptance in Europe and are slowly being reintroduced into the United States, primarily through imported passenger cars. It is possible that antilock brake systems for trucks will become common in the United States (or may be required by regulation) within 5 to 10 years. Thus, the improvements in truck braking distances that might result from antilock brake systems need be considered in the development of highway design criteria for future application.

The purpose of antilock brakes is to take full advantage of the available tire-pavement friction capabilities without locking the wheels and losing vehicle control. Antilock brake systems try to achieve and maintain the peak coefficient of tire-pavement friction shown in figure 1, thereby maximizing the braking effort.

Antilock brake systems operate by monitoring each wheel for impending lock up. When wheel lock up is anticipated, the system releases brake pressure on the wheel. When the wheel begins to roll freely again, the system reapplies braking pressure. The system constantly monitors each wheel and readjusts the brake pressure until the wheel torque is no longer sufficient to lock the wheel. The antilock brake system is controlled by an onboard microprocessor.

A recent NHTSA study of the performance of a commercially available antilock brake system on a two-axle single-unit truck found a 15 percent reduction in braking distance for a straight line stop from 60 mi/h (97 km/h) on a wet polished concrete pavement surface with an SN_{40} of approximately 30 (very similar to the surface used by the AASHTO Green Book in the specification of stopping sight distance standards); tests on other pavement surfaces and in other types of maneuvers found decreases in braking distance up to 42 percent with the antilock brake system.[21] Braking tests conducted by NHTSA for this study found improvements of 20 to 30 percent with use of an antilock braking system for straight line stops from 35 and 40 mi/h (56 and 64 km/h) by an empty tractor-trailer truck on a wet-pavement with SN_{40} equal to approximately 30 (see appendix A). Furthermore, in addition to improving the braking efficiency by operating closer to the peak braking friction coefficient, antilock brake systems should also minimize the increase in braking distance due to driver inexperience (see discussion in the following section).

d. Driver Control Efficiency

Most truck drivers have little or no practice in emergency braking situations. This lack of expertise in modulating the brakes results in braking distances that are longer than the vehicle capability. NCHRP Report 270 evaluated the effect of driver efficiency on braking distance using both experienced test drivers and professional truck drivers without test track experience.[12] The study found that the driver efficiencies ranged from 62 to 100 percent of the vehicle capability. The braking performance of the drivers tended to improve during the testing period as the drivers gained experience in modulating the brakes. Because so many drivers on the road lack experience in emergency braking, the study recommended the use of a driver efficiency of 62 percent in stopping sight distance design criteria. However, it should be recognized that this is a very conservative choice. The best-performance drivers can operate at efficiencies approaching 100 percent. Furthermore, in the future, antilock brake systems could eliminate the concern over driver efficiency by providing computer-controlled modulation of the brakes to achieve minimum braking distance.

4. Braking Distances for Use in Highway Design Criteria

NCHRP Report 270 has suggested a model to predict braking distance as a function of pavement surface characteristics, tire characteristics, vehicle braking performance and driver control efficiency.[12] Parametrically, the model expresses the coefficient of rolling friction, f_r, as:

$$f_r = f_p \times TF \times BE \times CE \tag{8}$$

where: f_p = Peak braking friction coefficient available given the pavement surface characteristics

TF = Adjustment factor for tire tread depth (see equation (7))

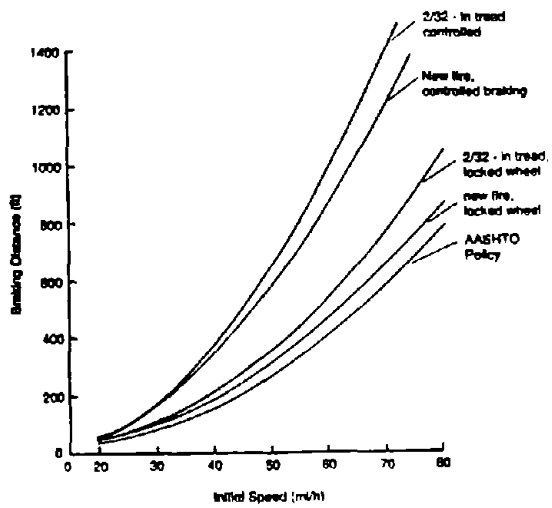
BE = Adjustment factor for braking efficiency (the efficiency of the braking system in using the available friction, typically 0.55 to 0.59 for conventional braking systems)

CE = Adjustment factor for driver control efficiency (the efficiency of the driver in modulating the brakes to obtain optimum braking performance, typically 0.62 to 1.00 for conventional braking systems)

The factors that influence each term of equation (8) have been addressed in the preceding discussion.

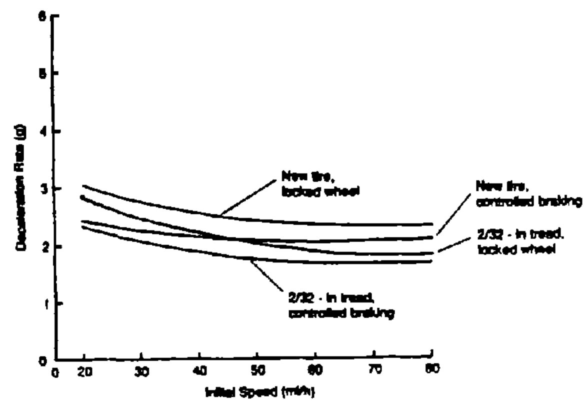
A paper by Fancher, derived from the NCHRP Report 270, used the model in equation (8) to predict truck braking distances.[12,19] Figure 3 shows the braking distances for trucks under controlled and locked wheel stops with new tires (15/32- to 31/32-in [2.4 to 4.9 cm] tread depth) and worn tires (2/32-in

[0.2 cm] tread depth) in comparison to the braking distances assumed in the AASHTO Green Book. Figure 3 shows that the braking distances predicted by Fancher are substantially longer than the distances for locked wheel braking by a passenger car assumed by AASHTO. The figure is based on a pavement with skid number (SN_{40}) of 28 and the best-performance driver who uses 100 percent of the vehicle braking capability. A less-experienced driver would require even longer stopping distances.



Note: 1 ft = 0.305 mi
1 mi = 1.61 km

Figure 3. Truck braking distances on a poor, wet road.[19]



Note: 1 mi = 1.61 km

Figure 4. Truck deceleration rates on a poor, wet road.[19]

Figure 4 illustrates the deceleration rates (i.e., values of f_r) used to develop figure 3. Figure 4 shows that the deceleration rates for controlled stops on a wet pavement by the best performance driver are generally between 0.20 and 0.25 g, and are relatively insensitive to vehicle speed. In contrast, appendix B of NCHRP Report 270 shows deceleration rates as high as 0.5 g in controlled stops on a wet pavement by some drivers.[12] These tests were performed on a pavement that apparently has a very high peak friction coefficient even when wet. The data in figures 3 and 4 were derived theoretically from the model given in equation (8).

The available literature does not provide a clear indication of which braking distances should be used in highway design criteria. Many of the factors that influence braking distances, such as pavement characteristics and driver efficiencies, vary widely. For purposes of the evaluation of current highway design and operational criteria in this report, three braking scenarios have been derived for consideration in the development of design criteria for trucks. These three scenarios are: tractor-trailer truck with a conventional brake system and the worst performance driver; tractor-trailer truck with a conventional brake system and the best performance driver; and, a tractor-trailer truck with an antilock brake system. Deceleration rates and braking distances for these three scenarios are shown in table 6. These data are based on the results shown in figures 3 and 4, with a minor change in the assumption concerning pavement surface properties (from SN_{40} of 28 assumed by Fancher to SN_{40} of 32 assumed by the AASHTO Green Book). All of the braking distances in table 6 are appropriate for an empty truck with relatively good radial tires (at least 12/32 in [1.0 cm] of tread depth).

The data for the worst performance driver in table 6 are based on an assumed 62 percent driver control efficiency [CE in equation (8)], which represents a very conservative, worst case condition. The data for the best performance driver are based on a driver control efficiency of 100 percent, and, thus, represent the full capability of conventional brake systems. Most truck drivers on the road today have control efficiencies that fall between these two extremes. The data for an antilock brake system represent deceleration rates between 0.31 and 0.36 g, which are consistent with the result of the NHTSA tests reported in appendix A. This may, in fact, be a conservative estimate of the improvement that could be obtained from future antilock brake systems.

It is important to note that the estimates of deceleration rate and braking distances in table 6 for trucks equipped with antilock brake systems are very similar to the AASHTO criteria for passenger cars, which are also shown in the table.

C. Driver Eye Height

Driver eye height is a combined driver and vehicle characteristic that is essential to the evaluation of sight distance issues. Truck drivers generally have substantially higher eye heights than passenger car drivers, which means that a truck driver can see farther than a passenger car driver at vertical sight restrictions.

The AASHTO Green Book and the MUTCD specify a value of 42 in (107 cm) for driver eye height, based on consideration of a passenger car as the design vehicle. This criterion was recently decreased from 45 in (114 cm) as a result of field studies of passenger car driver eye heights.

A 1983 FHWA study examined data from the literature as well as information provided by truck manufacturers.[22] This study concluded that the driver eye heights shown in table 7 represented average values for the specified truck tractors.

Table 6. Truck deceleration rates and braking distances for use in highway design.[a]

Vehicle speed (mi/h)	Deceleration rate (g)				Braking distance (ft)			
	AASHTO policy	Worst-performance driver[b]	Best-performance driver[c]	Antilock brake system	AASHTO policy	Worst-performance driver[b]	Best-performance driver[c]	Antilock brake system
20	0.40	0.17	0.28	0.36	33	77	48	37
30	0.35	0.16	0.26	0.34	86	186	115	88
40	0.32	0.16	0.25	0.31	167	344	213	172
50	0.30	0.16	0.25	0.31	278	538	333	269
60	0.29	0.16	0.26	0.32	414	744	462	375
70	0.28	0.16	0.26	0.32	583	1,013	628	510

[a] Based on an empty tractor-trailer truck on a wet pavement with SN_{40} = 32.
[b] Based on driver control efficiency of 0.62.
[c] Based on driver control efficiency of 1.00.
Note: 1 mi = 1.61 km
1 ft = 0.305 m

Table 7. Average driver eye heights for trucks.[22]

Tractor type	Average driver eye height (in)
Conventional cab	93
Cab over engine	107
Low cab over engine	91

Note: 1 in = 2.54 cm

In addition, the 1983 FHWA study found that truck driver eye heights have a very large range, not only between tractor types and models, but also between different drivers for the same tractor. Even for a given tractor, driver eye height is a function of driver characteristics (typically, a range from the 5th percentile female to the 95th percentile male is used), seat adjustment, tires, suspension, and load.

As an example, data obtained in the 1983 FHWA study discussed above indicated an estimated range of possible eye heights from 71.5 to 100 in (182 to 254 cm) for IH S-Series tractors. The IH 42/4300 series conventional cabs had a range of eye heights from 87 to 101 in (221 to 257 cm) and their CO-9670 cab-over-engine tractors had a range of eye heights from 94.5 to 112.5 in (240 to 286 cm). Data for Mack tractors were also examined; these data showed a smaller range because of a fixed seat position (and, possibly, only a single individual was considered). Freightliner provided only single values of driver eye height for their trucks.[22]

A 1982 NHTSA study examined the eye heights of 16 tractors representing four manufacturers.[23] Single values of driver eye height were reported for each tractor, ranging from 80.9 to 112.5 in (205 to 286 cm).

A 1978 FHWA study evaluated tractors by three manufacturers and reported average driver eye heights of 101 in (257 cm) for conventional tractors and 94 in (239 cm) for cab-over-engine tractors.[24] Possibly the latter data represent low cab-over-engine tractors, since both the 1983 FHWA study and common sense indicate that cab-over-engine tractors should have greater driver eye heights than conventional tractors.[22]

Based on the available data, the range of driver eye heights for today's truck fleet and driver population is estimated as from 71.5 to 112.5 in (182 to 286 cm). Analyses of sight distance criteria later in this report consider driver eye heights of 75 in or 191 cm (a low, but not extreme, value) and 93 in or 236 cm (a typical average value).

D. Truck Acceleration Characteristics

Two aspects of truck acceleration performance are considered in this section. The first aspect is the ability of a truck to accelerate from a full stop to clear a specified hazard zone such as an intersection or railroad-highway grade crossing. Typically, a hazard zone of this type is less than 200 ft (66 m) long; as a result, the speed attained by the truck is low, usually between 10 and 15 mi/h (16 and 24 km/h). This first aspect of truck acceleration performance is, therefore, referred to as "low-speed acceleration." The second aspect of truck acceleration is the ability of a truck to accelerate to a high speed either from a stop or from a lower speed. This type of acceleration, referred to here as "high-speed" acceleration, is needed by trucks in passing maneuvers and in entering a high-speed facility.

1. Low-Speed Acceleration

The low-speed (or start-up) acceleration ability of a truck determines the time required for it to clear a relatively short hazard zone such as an intersection or railroad-highway grade crossing. The primary factors that affect the clearance times of trucks are:

- Length of hazard zone.
- Length of truck.
- Truck weight-to-power ratio.
- Truck gear ratio.
- Roadway geometry (percent grade, curvature).

A fairly complex mathematical model, including the effects of truck engine and transmission parameters, is necessary to adequately represent all of the variables involved in the low-speed acceleration of trucks. The development of such a model has been reported in the literature but the model itself is no longer available.[25]

A simplified analytical model of the low-speed acceleration of trucks has been developed by Gillespie.[25] The Gillespie model estimates the time required for a truck to clear a hazard zone, starting from a full stop, as:

$$t_c = \frac{0.682\ (L_{HZ} + L_T)}{V_{mg}} + 3.0 \qquad (9)$$

where: t_c = time required to clear hazard zone (s)

L_{HZ} = length of hazard zone (ft)

L_T = length of truck (ft)

V_{mg} = maximum speed in the gear selected by the driver (mi/h)

Equation (9) is based on the assumption that the distance traveled by the truck during the clearance time is the length of the hazard zone plus the length of the truck, $L_{HZ} + L_T$. Neither the weight or the weight-to-power ratio of the truck is considered explicitly in equation (9), although it is implicitly assumed that the weight-to-power ratio would affect the driver's choice of gears. On a level road, V_{mg} can be calculated as:

$$V_{mg} = \frac{60}{gr} \tag{10}$$

where: gr = gear ratio selected by the driver

This model of low-speed acceleration is based on the assumption that the gear design, engine speed, and tire size of the truck are such that its maximum speed is 60 mi/h (97 km/h). It is also assumed that the truck will remain in its initial gear through the entire hazard zone. State and Federal regulations require vehicles transporting passengers and hazardous materials to accelerate at railroad-highway grade crossings without shifting gears. The assumption that the truck does not shift gears is probably less realistic at intersections than at railroad-highway grade crossings. When shifting gears is allowed, a truck has the potential to reach a higher speed but, at the same time, it loses speed during the delay when the driver is shifting gears. Therefore, the overall effect on clearance time (t_c) of assuming that there is no gear shift may be negligible unless the hazard zone is quite long.

The estimated clearance times for a 65-ft (19.8 m) tractor-trailer truck, obtained from equation (9), are given in table 8. The values of clearance times on grades are obtained by multiplying the clearance time on a level road by a grade factor, F_g. The values of F_g derived by Gillespie are:[25]

Percent Grade	3-5	6-10	11-13
Grade Factor (F_g)	1.26	1.47	1.78

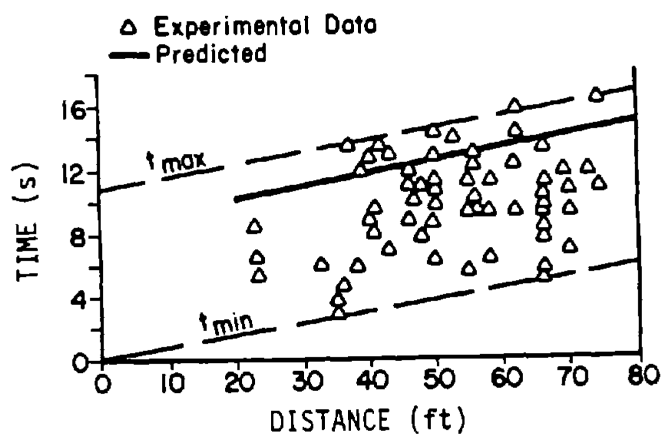
It should be noted that the results of the low-speed acceleration analysis in table 8 are presented in terms of clearance times rather than in terms of acceleration rates. The model assumes that, when starting from a full stop, a truck rather quickly reaches the maximum speed in the gear selected by the driver and then travels at that constant speed until it clears the hazard zone. Thus, equation (9) is essentially a constant speed model and acceleration rates, as such, are not meaningful.

The Gillespie model was compared with the results of field observations of time versus distance for 77 tractor-trailer trucks crossing zero-grade intersections from a full stop.[25] These data are shown in figure 5. There is no information on the weights or weight-to-power ratios of these trucks although they probably vary widely. A line representing the clearance time predicted by equation (9) for a level grade is also presented in the figure. Equation (9) provides a relatively conservative estimate of clearance times, since the majority of the experimental points fall below the prediction.

Table 8. Clearance time (s) for low-speed acceleration by a tractor-semitrailer.

Percent grade	V_{mg} (mi/h)	Length of hazard zone (ft) 30	40	50	60	70	80	90	100	110	120
0-2	8	11.1	11.9	12.8	13.7	14.5	15.4	16.2	17.1	17.9	18.8
3-5	6	13.8	14.9	16.1	17.2	18.3	19.5	20.6	21.8	22.9	24.0
6-10	5	16.0	17.3	18.7	20.0	21.4	22.8	24.1	25.5	26.9	28.2
11-13	4	19.2	20.9	22.6	24.3	26.0	27.7	29.4	31.1	32.8	34.5

Note: 1 mi = 1.61 km
1 ft = 0.305 m



Note: 1 ft = 0.305 m

Figure 5. Field observations of time for tractor-trailer trucks to clear intersection after starting from a stop.[25]

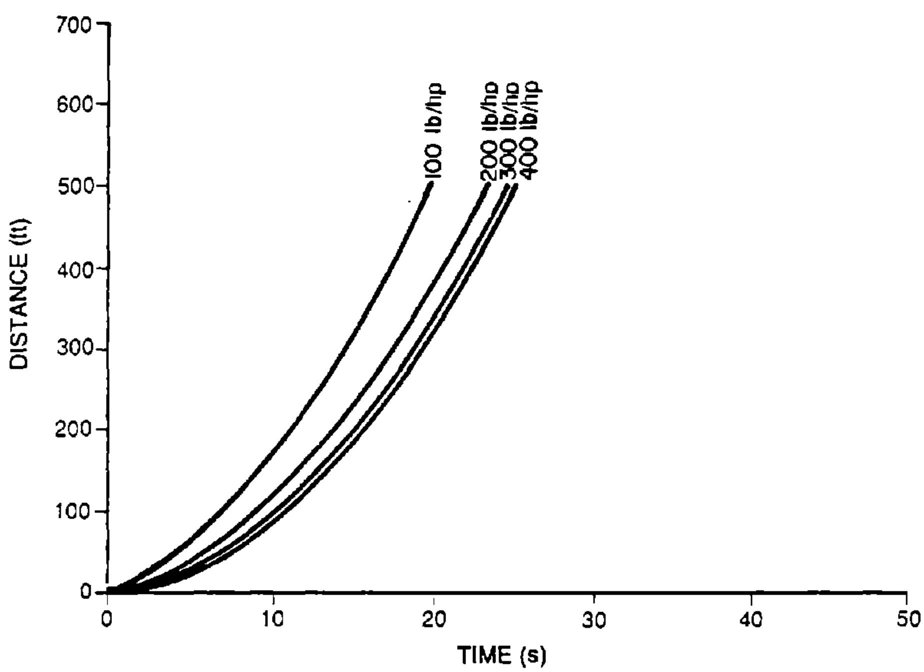
The experimental data in figure 5 can be bounded by two lines representing the maximum and minimum observed clearance times. The equations representing these lines are:

$$t_{min} = 0.075\ (L_{HZ} + L_T) \tag{11}$$

$$t_{max} = 10.8 + 0.075\ (L_{HZ} + L_T) \tag{12}$$

Notice that the lines for t_{min} and t_{max} are parallel.

Hutton collected data on the acceleration performance of 31 tractor-trailer combinations.[26] The majority of the trucks evaluated by Hutton were cab-over-engine tractors pulling twin 27-ft (8.2-m) trailers. The engine horsepower of the trucks ranged from 228 to 375 hp (170 to 283 kW), while their gross weights ranged from 33,250 to 89,900 lb (15,100 to 40,900 kg). Figure 6 illustrates the resulting time versus distance curves determined by Hutton for initial acceleration by trucks with weight-to-power ratios of 100, 200, 300, and 400 lb/hp (0.06, 0.12, 0.18, and 0.24 kg/W). The following equations provide an analytical representation of the curves in figure 6.

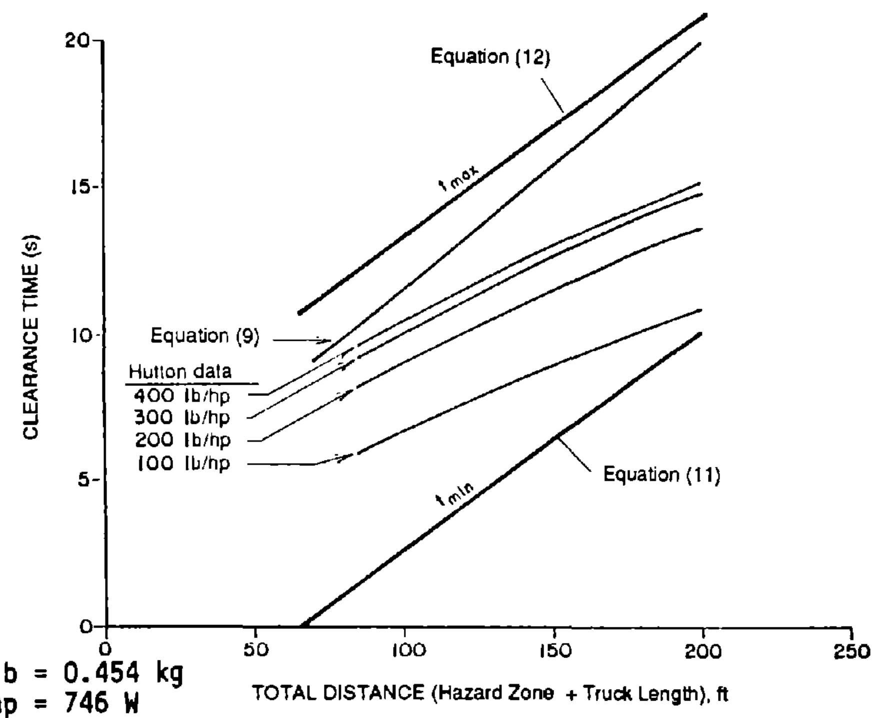


Note: 1 lb = 0.454 kg
1 hp = 746 W

Figure 6. Observed time versus distance curves for initial acceleration from a stop by tractor-trailer trucks.[26]

Weight-to-power ratio	Clearance time (t_c) (s)
100	$-6.0 + \sqrt{36 + 1.25\ (L_{HZ} + L_T)}$
200	$-3.2 + \sqrt{10.2 + 1.40\ (L_{HZ} + L_T)}$
300	$-1.9 + \sqrt{3.8 + 1.40\ (L_{HZ} + L_T)}$
400	$-0.6 + \sqrt{0.4 + 1.25\ (L_{HZ} + L_T)}$

Figure 7 compares the clearance times based on the Hutton data with those based on the Gillespie data. The Hutton data fall within the extreme boundaries (t_{min} and t_{max}) established for the Gillespie data. Equation (9) still provides a conservative estimate of clearance time, since all of the Hutton data fall below it. The line of maximum clearance time (t_{max}) exceeds the 400 lb/hp (0.24 kg/W) line by approximately 30 percent. Since Hutton reported considerable scatter in his experimental data, primarily due to different driving skills, it seems reasonable to allow a 30 percent margin around the Hutton data for driver variations. Since the line representing the minimum clearance time (t_{min}) in the Gillespie data is well below the Hutton data for a 200-lb/hp (0.12 kg/W) truck, it seems reasonable to move the lower bound for clearance time to 70 percent of the observed average clearance time for a 100-lb/hp (0.06 kg/W) truck.



Note: 1 lb = 0.454 kg
1 hp = 746 W
1 ft = 0.305 m

Figure 7. Comparison of time for a tractor-trailer truck to clear an intersection starting from a stop based on Gillespie and Hutton data.[25,26]

Thus, the recommended range for clearance times for trucks has been revised as follows:

$$t_{min} = -4.2 + 0.70 \sqrt{36 + 1.25\ (L_{HZ} + L_T)} \tag{13}$$

$$t_{max} = 10.8 + 0.075\ (L_{HZ} + L_T) \tag{14}$$

Table 9 presents the estimated minimum and maximum clearance times for a 65-ft (19.8-m) truck to cross hazard zones of varying length.

Table 9. Minimum and maximum clearance times (s) for 65-ft (19.8-m) tractor-trailer truck.

Range of clearance Times	Length of hazard zone (ft)									
	30	40	50	60	70	80	90	100	110	120
t_{min}	4.5	4.9	5.2	5.5	5.8	6.1	6.4	6.7	7.0	7.2
t_{max}	17.9	18.7	19.4	20.2	20.9	21.7	22.4	23.2	23.9	24.7

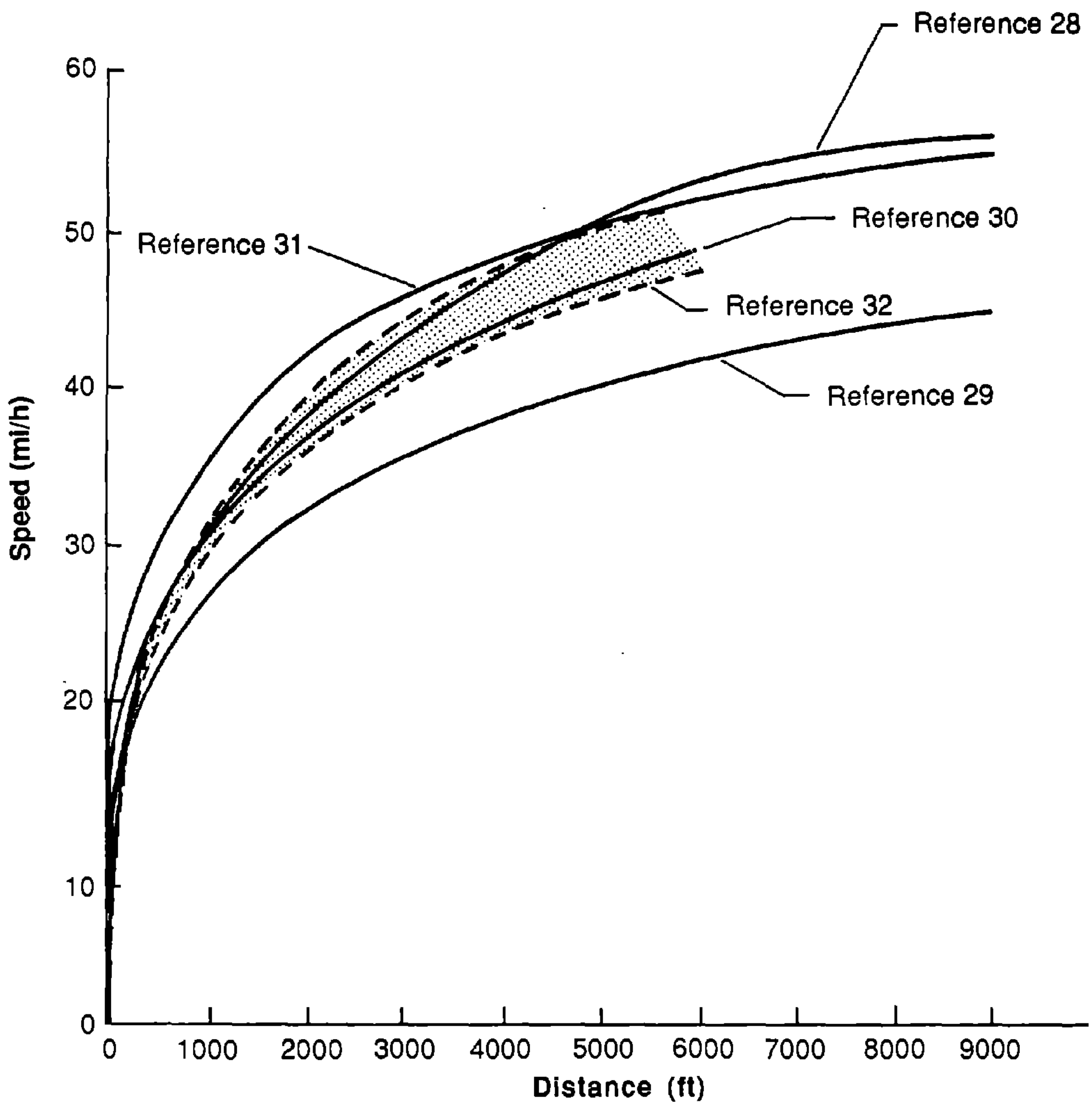
Note: 1 ft = 0.305 m

Fancher compared the results of two studies to the time versus distance for low-speed acceleration from a stop specified by AASHTO and found that the average tested heavy vehicles performed with more acceleration than the AASHTO criteria for a WB-50 truck.[27]

2. High-Speed Acceleration

There is a substantial amount of performance data in the literature for acceleration from a stop to a high speed. Figure 8 presents speed versus distance curves for acceleration to high speeds developed in references 28, 29, 30, 31, and 32. All of these sources are at least 10 years old and reflect the performance of past truck populations.

Hutton also developed acceleration data for trucks classified by weight-to-power ratio.[26] Although these data were collected in 1970, the fundamental relationships between weight-to-power ratio and truck performance may not have changed substantially.



Note: 1 mi = 1.61 km
1 ft = 0.305 m

Figure 8. Speed versus distance curves for truck acceleration from a stop.[33]

Figure 9 shows distance versus time curves for acceleration from a full-stop to higher speeds for 100, 200, 300, and 400 lb/hp (0.06, 0.12, 0.18, and 0.24 kg/W) trucks. These curves can be approximated by the following analytical relationships:

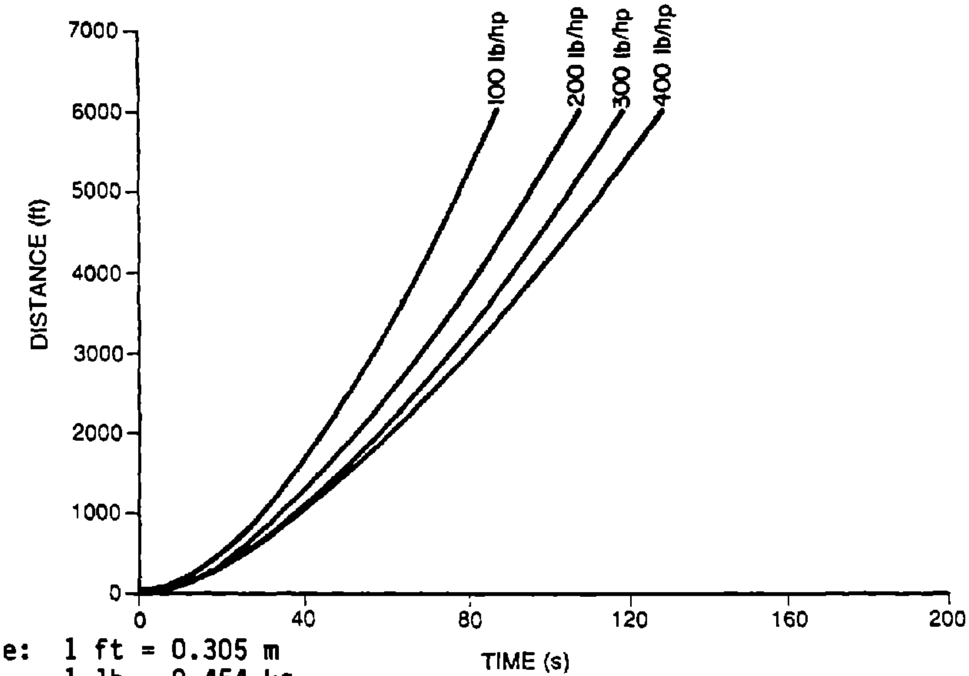
Weight-to-power ratio (lb/hp)	Distance-time relationship	
100	$t = -15.1 + \sqrt{229 + 1.64\ x}$	(15)
200	$t = -22.8 + \sqrt{523 + 2.56\ x}$	(16)
300	$t = -22.0 + \sqrt{480 + 2.94\ x}$	(17)
400	$t = -26.6 + \sqrt{708 + 3.57\ x}$	(18)

Figure 10 shows speed versus time curves for the same trucks shown in figure 9. The average acceleration rates for acceleration to 40 mi/h (64 km/h) from speeds of 0, 10, 20, and 30 mi/h (0, 16, 32, and 48 km/h) are given in table 10, based on the data in figure 10. Acceleration rates of trucks at higher speeds are less than those given in table 10. For example, the acceleration rate for a 100-lb/hp (0.06-kg/W) truck to increase its speed from 35 to 55 mi/h (56 to 88 km/h) is 0.53 ft/s² (0.16 m/s²), based on the curve in figure 10. The corresponding rate for a 200-lb/hp (0.12-kg/W) truck is 0.36 ft/s² (0.11 m/s²). Figure 10 illustrates that 300- and 400-lb/hp (0.18- and 0.24-kg/W) trucks cannot accelerate to 55 mi/h (88 km/h) within the time scale shown on the figure.

E. Speed-Maintenance Capabilities on Grades

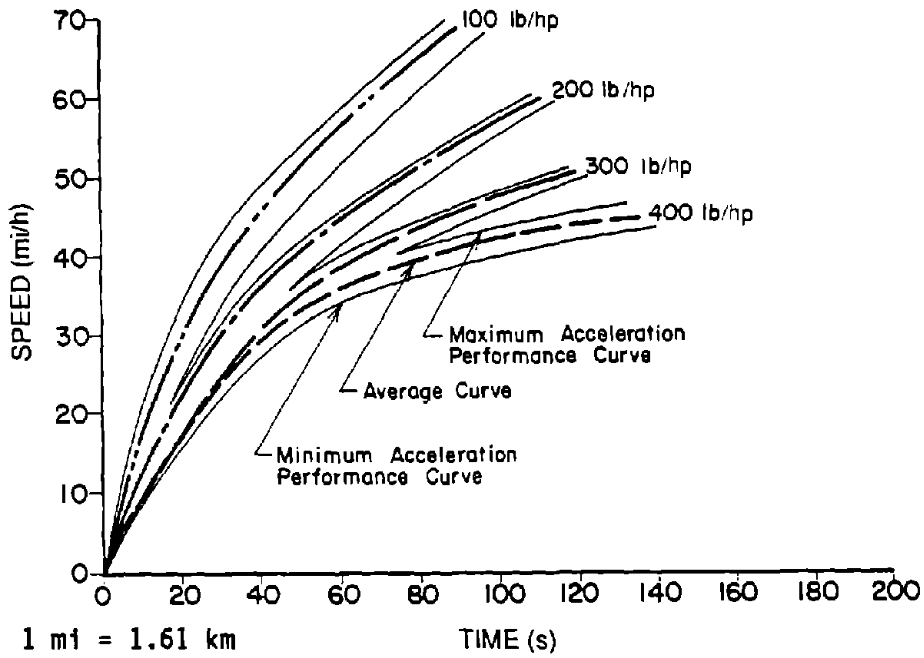
The primary factors that determine the ability of a truck to maintain speed on an upgrade are:

- Weight-to-power ratio.
- Rolling resistance.
- Aerodynamic drag.
- Transmission characteristics.
- Tire size.
- Drive line efficiency.
- Percent grade of roadway.
- Length of grade.



Note: 1 ft = 0.305 m
1 lb = 0.454 kg
1 hp = 746 W

Figure 9. Observed time versus distance curves for acceleration to high speed from a stop by a tractor-trailer truck.[26]



Note: 1 mi = 1.61 km
1 lb = 0.454 kg
1 hp = 746 W

Figure 10. Observed speed versus time curves for acceleration by trucks with various weight-to-power ratios.[26]

Table 10. Average acceleration capabilities of trucks from specified speed to 40 mi/h (64 km/h).[26]

Weight-to-power ratio (lb/hp)	Acceleration rate (ft/s²)			
	0 mi/h	10 mi/h	20 mi/h	30 mi/h
100	1.87	1.70	1.47	1.29
200	1.22	1.08	0.96	0.79
300	0.91	0.81	0.72	0.58
400	0.71	0.61	0.50	0.36

Note: Based on speed-distance curves shown in figure 10.
1 lb = 0.454 kg
1 hp = 746 W
1 ft = 0.305 m
1 mi = 1.61 km

The speed of a truck on an upgrade is governed by the following equation:

$$m\dot{V} = P/V - F_r - F_a - mg \sin\alpha \tag{19}$$

where:
- m = mass of truck
- P = net engine power available at the drive wheels (hp)
- V = speed (ft/sec)
- F_r = rolling resistance force (lb)
- F_a = aerodynamic drag force (lb)
- α = angle of the grade (degrees)
- g = acceleration of gravity (32.2 ft/s² or 9.8 m/s²)

The steepness of grade can be expressed in the more conventional percent grade form as 100 tan α. The $\dot{V}$ term represents the time derivative of truck speed (dV/dt).

Equation (19) can also be written as:

$$m\dot{V} = \frac{mg}{(W/P)V} - F_r - F_a - mg \sin\alpha \tag{20}$$

where W/P is weight-to-power ratio in units of lb/hp. Several of the key factors in equations (19) and (20) are discussed below.

1. Literature Review

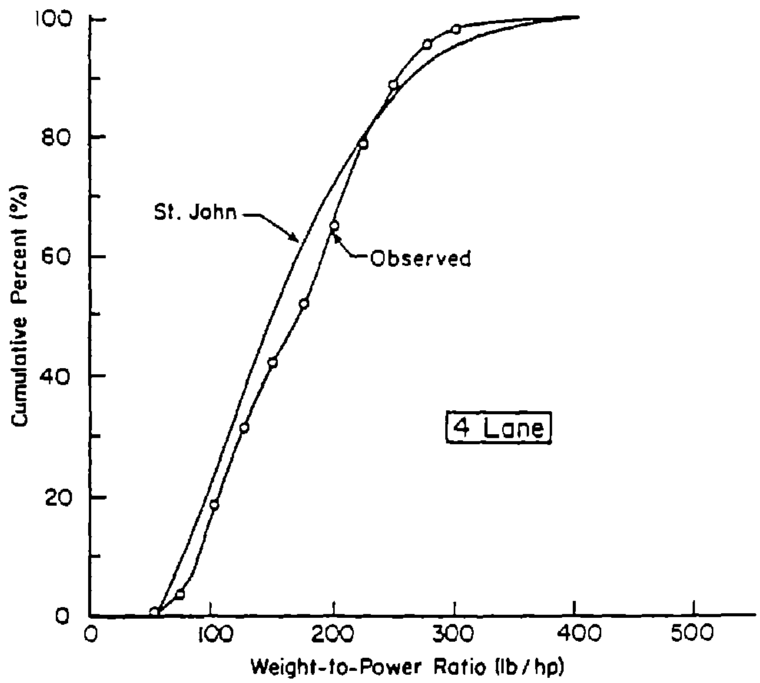
a. Weight-to-power ratio

The ability of a truck to maintain speed on an upgrade is very sensitive to its weight-to-power ratio. The weight-to-power ratios of trucks have been decreasing steadily for the last 20 years, as tractor engines have become more and more powerful.

A number of studies have addressed recent trends in the weight-to-power ratios of trucks. Figure 11 shows an estimate of the distribution of weight-to-power ratios of trucks made by St. John in 1979 from data collected by the FHWA Bureau of Motor Carrier Safety and the California Department of Transportation.[34,35] The figure shows that St. John's distribution compares well with field data reported by Messer in a 1983 study.[36] The St. John and Messer data are in agreement that the median weight-to-power ratio of trucks is about 160 lb/hp (0.10 kg/W) and the 15th percentile weight-to-power ratio (at the poor end of the performance distribution) is about 240 lb/hp (0.15 kg/W).

Table 11 presents average values of weight-to-power ratio of trucks obtained from field observations at sites located in the Eastern and Western parts of the United States in a 1985 study by Gillespie.[37] The table shows the average weight, power, and weight-to-power ratios of trucks by truck type and road class. The number of trucks observed for each road class is given in parentheses following the road class.

Figure 12 illustrates the long-term trends in the weight-to-power ratios of trucks. The figure shows the several lines illustrating trends in average weight-to-power ratio of trucks as a function of gross weight from 1949 to 1975, based on figure III-27 in the AASHTO Green Book. Added to the figure is a line based on the 1977 Truck Inventory and Use Survey (TIUS) performed by the Bureau of the Census and points representing the 1985 Gillespie data.[37,38] The TIUS is a survey of the owners of a nationwide representative sample of approximately 120,000 registered trucks conducted at 5-year intervals. The TIUS includes both light trucks (pickups) and heavy trucks with gross vehicle weights over 10,000 lb (4,540 kg), although the light trucks have been excluded from the data shown in Figure 12. The figure shows that the long-term decrease in weight-to-power ratios of trucks has continued right up to the present. A comparison of the TIUS and Gillespie data demonstrate that the major reason for the reduced weight-to-power ratios of trucks over the last decade is a substantial increase in average engine horsepower. The average tractor power in the 1977 TIUS data was 282 hp (0.17 kg/W), in comparison to 350 hp (0.21 kg/W) in the Gillepsie data.



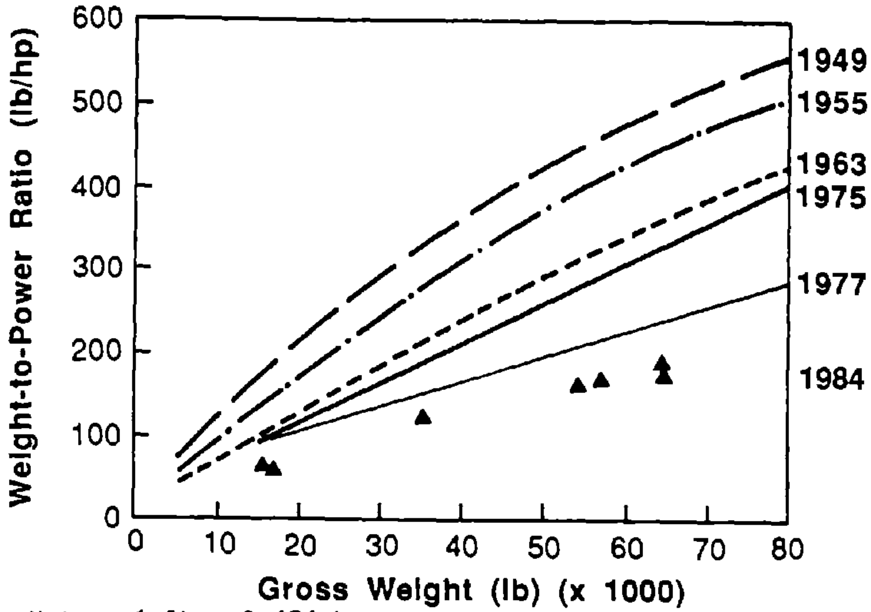
Note: 1 lb = 0.454 kg
1 hp = 746 W

Figure 11. Comparison of weight-to-power ratios estimated by St. John to those observed by Messer.[35,36]

Table 11. Average weights and power values for trucks.[37]

	Weight (lb)	Power (hp)	Weight/Power
Straight Trucks			
Interstate - East (14)	15,233	219	70
Interstate - West (6)	35,050	267	131
Primary - East (6)	16,575	273	75
Tractor-Trailers			
Interstate - East (157)	54,452	328	166
Interstate - West (233)	64,775	370	175
Primary - East (134)	57,487	330	174
65-ft Doubles			
Interstate - West (19)	64,920	331	196

Note: 1 lb = 0.454 kg
1 hp = 746 W



Note: 1 lb = 0.454 kg
1 hp = 746 W

Figure 12. Trend in weight-to-power ratios of trucks from 1949 to 1984.[1,37,38]

b. Rolling resistance

The rolling resistance of tires, F_r, is defined as the ratio of power lost due to rolling resistance to speed. F_r can be estimated using the following SAE equations:

$$F_r = 0.001\ (4.1 + 0.041\ V) \text{ for radial tires} \tag{21}$$

$$F_r = 0.001\ (5.3 + 0.044\ V) \text{ for mixed tires} \tag{22}$$

$$F_r = 0.001\ (6.6 + 0.046\ V) \text{ for bias-ply tires} \tag{23}$$

where V is speed in mi/h. Experimental rolling resistance data for selected truck tires can be found in the literature.[39]

c. Aerodynamic drag

The aerodynamic drag force is estimated by the following relationship:[30]

$$F_a = 1.1D\ C_D\ A\ V^2 \tag{24}$$

where: F_a = aerodynamic drag (lb)

D = air density (lb/ft^3)

C_D = drag coefficient (0.6 with aerodynamic aids, 0.7 without)

A = truck frontal area (102 ft^2 for van bodies, 75 ft^2 for cab only) (ft^2)

V = truck speed (mi/h)

2. Reanalysis of Gillespie Data

The most recent data on truck performance on grades were those collected by Gillespie in 1984. Since the reported results did not include the explicit distribution of weight-to-power ratios, the data base developed in that study was obtained and reanalyzed by the authors. Appendix C in volume II presents a detailed discussion of the procedures used to derive weight-to-power ratios for over 3,000 individual trucks theoretically from their final climbing speeds and directly from the weights and rated horsepowers of a sample of approximately 500 trucks. This analysis addressed only combination trucks (tractor-trailers) and addressed several factors including aerodynamic losses that were not addressed by Gillespie. The distributions of truck weight-to-power ratio were derived indirectly from the final climbing speeds and directly from measured gross weights and rated horsepowers. These distributions showed that the median weight-to-power ratio for trucks is about 175 lb/hp (0.10 kg/W), while the 87.5 percentile weight-to-power ratio is about 250 lb/hp (0.15 kg/W).

F. Turning Radius and Offtracking

The minimum turning radius of a truck is defined as the path of the outer front wheel, following a circular arc, at a speed of less than 10 mi/h (16 km/h), and is limited by the vehicle steering mechanism. The dimensions and turning radii of three current AASHTO design vehicles are shown in table 12.

Table 12. Dimensions and turning radii of current AASHTO design vehicles.[1]

Design vehicle	Width (ft)	Wheelbase (ft)	Minimum turning radius (ft)
WB-40	8.5	40	40
WB-50	8.5	50	45
WB-60	8.5	60	45

Note: 1 ft = 0.305 m

When any vehicle is making a turn, its rear wheels do not follow the same path as its front wheels. The magnitude of this difference in paths, known as "offtracking," increases with the vehicle wheelbase and decreases with the radius of turn. Offtracking by passenger cars is minimal because of their relatively short wheelbases; however, many trucks offtrack substantially. The most appropriate descriptor of offtracking for use in highway design is the "swept path width," shown for a tractor-trailer in figure 13 as the difference in paths between the outside front tractor tire and the inside rear trailer tire.

The turning radius and offtracking of trucks are important design considerations for intersections and horizontal curves. Complete discussions of the role of offtracking in the design of intersections and horizontal curves are found in sections III-E and III-L of this report, respectively.

G. Suspension Characteristics

This section of the report reviews the characteristics of truck suspensions. The review is based primarily on a summary of suspension characteristics from the NHTSA factbook of truck characteristics.[4] Other references are cited in the text as appropriate.

The suspension of a heavy vehicle affects its dynamic responses in three major ways:

- Determining dynamic loads on tires.
- Orienting the tires under dynamic loads.
- Controlling vehicle body motions with respect to the axles.

Suspension characteristics can be categorized by eight basic mechanical properties:

- Vertical stiffness.
- Damping.
- Static load equalization.
- Dynamic inter-axle load transfer.
- Height of roll center.
- Roll stiffness.
- Roll steer coefficient.
- Compliance steer coefficient.

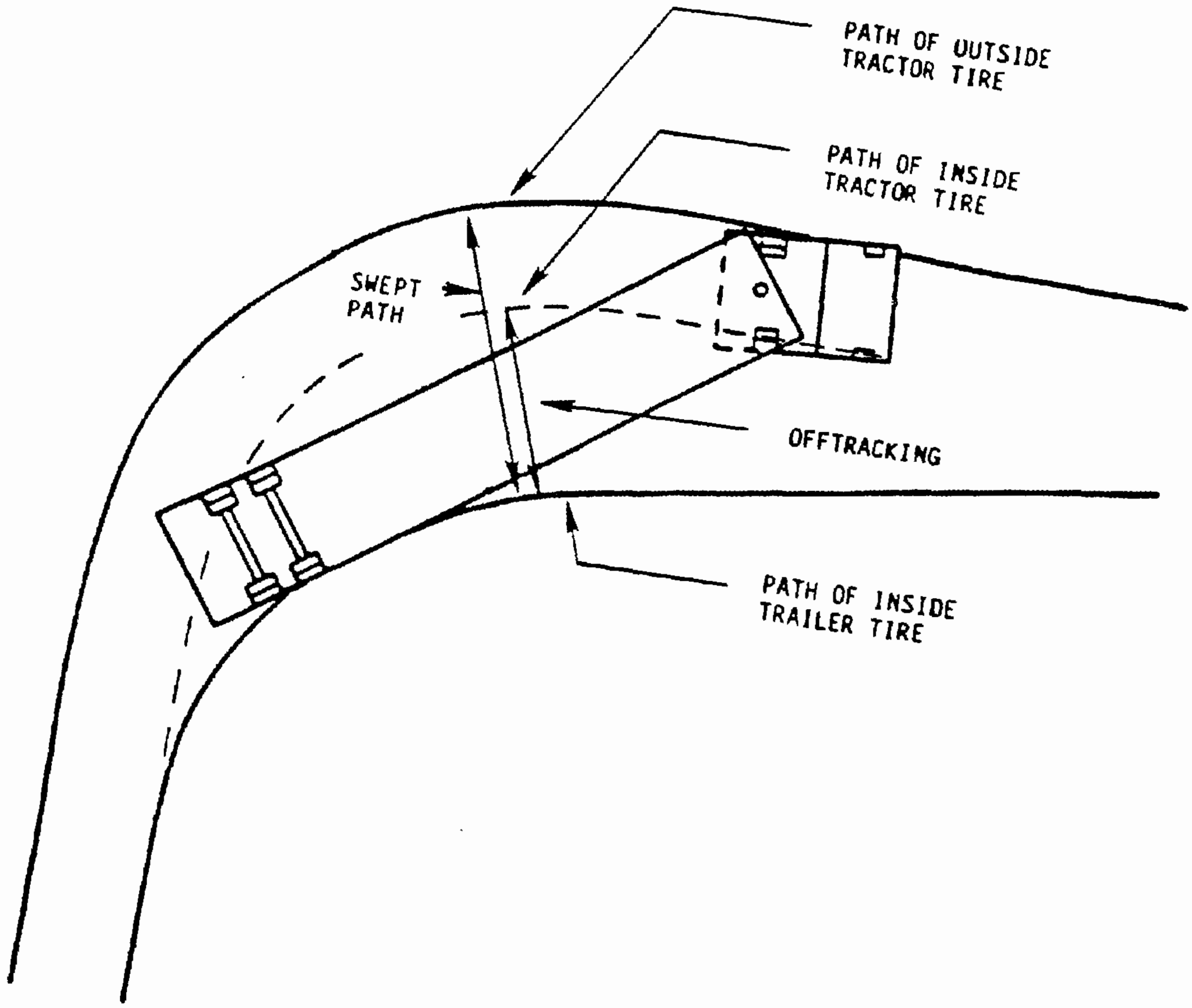


Figure 13. Swept path width and offtracking of a truck negotiating a 90° intersection turn.[40]

These suspension characteristics are important in determining the stability of trucks on horizontal curves.

The three most common types of tandem axle suspensions include two leaf-spring suspensions (known as the walking-beam and four-spring suspensions), and air-spring suspensions. Drawings of a walking-beam and four-spring suspension are presented in figures 14 and 15, respectively. Air-spring suspensions need shock absorbers, air plumbing, air valves, and anti-roll mechanisms that make their initial cost higher than leaf-spring suspensions.[41] However, air-spring suspensions offer a number of advantages over leaf-spring suspensions. These advantages include the availability of height regulating valves, significantly adjustable spring constants, and maintenance of a constant natural frequency over a wide range of loads.[41]

1. <u>Vertical Stiffness</u>: Dependent on spring stiffness.

The vertical stiffness of a truck suspension is mainly determined by the spring elements. Generally these elements are either leaf springs or air springs.

The vertical loads on the tandem axle of the trailer of a loaded truck can be up to four times greater than when the tractor is unloaded.[41] Since the load on the suspension can vary greatly, the springs must be very stiff for a fully loaded truck and much less stiff for an unloaded truck. Air springs are particularly well suited for such a range of loadings, because the spring rate can change significantly with loading. With leaf springs the stiffness can also change under different loadings, but not quite as much as for the air suspension. This creates a poor ride quality for unloaded conditions. The friction of leaf springs has an effect on its force-displacement relationship. (See the subsequent discussion of damping.)

Vertical stiffness is extremely important in ride quality, and it also has some effect on vehicle dynamics. A truck tends to pitch when the brakes are applied. As the truck pitches, weight is transferred from the rear axles to the front axles, and back again. This has a minor effect on braking efficiency and does not warrant alteration of the suspension design, particularly since ride quality might suffer.

The range of vertical stiffness for the various types of suspensions has been measured for a load of 10,000 lb (4,500 kg) on the front axles and 16,000 lb (7,300 kg) on the rear axles. The range of vertical stiffness per axle is given in table 13.

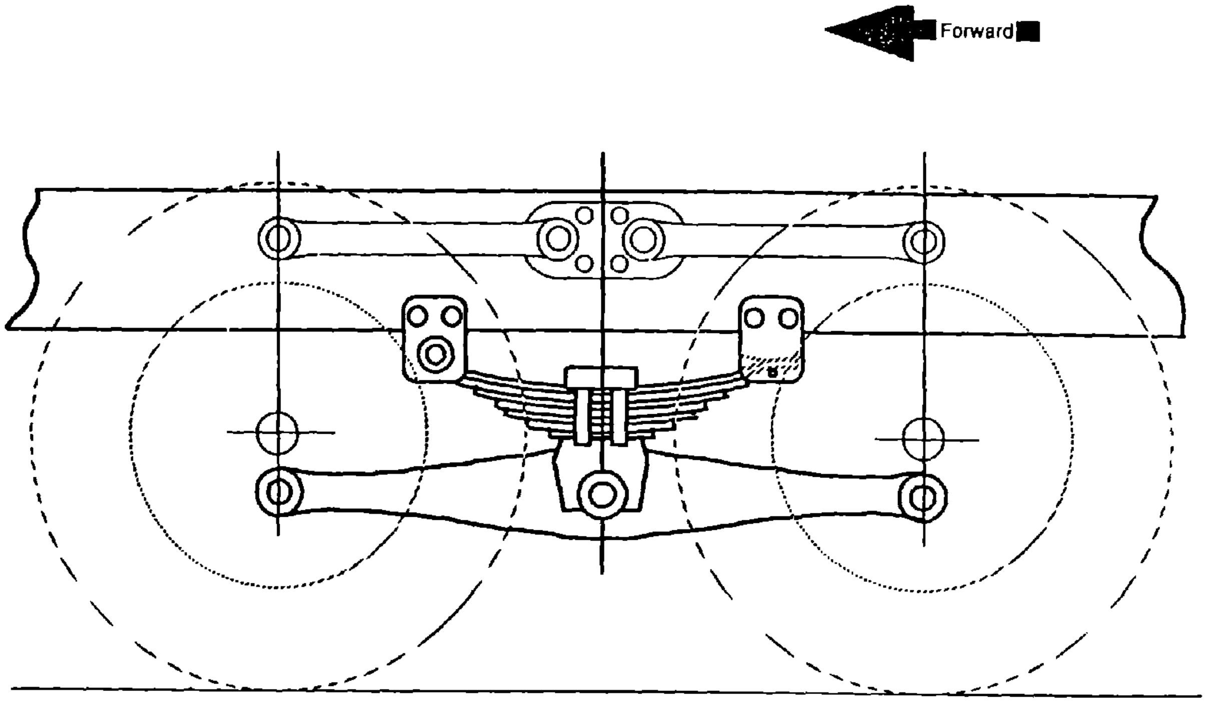


Figure 14. Diagram of walking-beam suspension.[4]

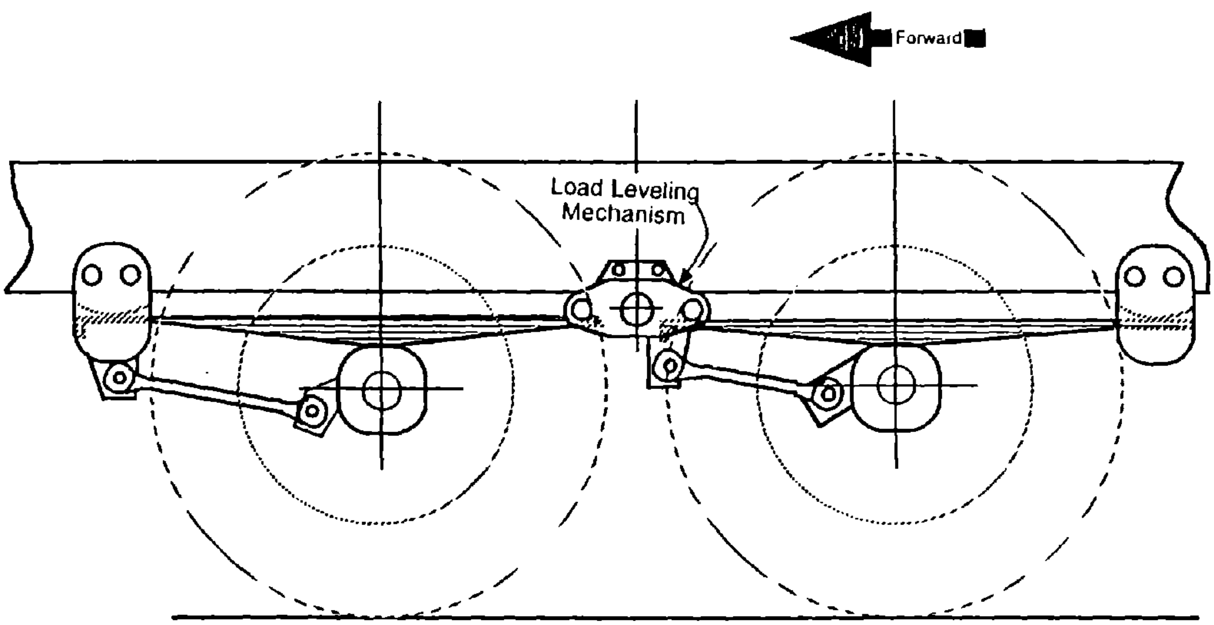


Figure 15. Diagram of four-spring suspension.[4]

Table 13. Typical range of vertical stiffness per axle for truck suspensions.[4]

Type of suspension	Range of vertical stiffness (lb/in)
Front suspension	2,000 - 2,750
Air suspension	1,000 - 7,000
Four-spring	8,000 - 21,000
Walking beam	10,000 - 21,000
Single-axle leaf spring	8,500 - 13,750

Note: 1 lb = 0.45 kg
1 in = 2.54 cm

2. Damping: Dependent on shock absorbers and coulomb friction of leaf springs.

Suspensions that have leaf springs rely on coulomb friction for damping. Coulomb friction comes from the rubbing at the interfaces of the various leaves of the spring. Therefore, the damping is a function of mean load and displacement. Air spring suspensions do not have the coulomb friction of leaf springs and, therefore, need shock absorbers to provide damping.

Damping has a moderate effect on rearward amplification and the transient dynamic behavior of the vehicle. A lack of damping can create a system that is likely to oscillate and produce large dynamic loads on the axles. Damping is set so that a maximum ride quality can be achieved. Increased damping usually reduces rearward amplification of steering inputs in multitrailer combination trucks and can, thus, increase stability in emergency maneuvers. A typical range of values for damping is given in table 14.

Table 14. Typical range of damping for truck suspension.[4]

Type of suspension	Range of damping (lb)
Front suspension	800 - 1,250
Air suspension	550 - 1,200
Four-spring	1,200 - 2,700
Walking beam	700 - 2,000
Single-axle leaf spring	1,800 - 2,400

Note: 1 lb = 0.454 kg

3. <u>Static Load Equalization</u>: Dependent on coulomb friction and mechanisms intended to evenly distribute loads on both axles on a tandem set.

Load equalization involves the design of tandem-axle suspensions to distribute the load equally between the two axles of the tandem. This type of load equalization is a static quantity; dynamic inter-axle load transfers are discussed in the next section.

Good load equalization is important for a truck traveling over a bumpy road, when the frame of the truck is being pitched. Tandem-axle suspensions, such as the walking-beam and four-spring suspensions, are typically made symmetrical to distribute the load evenly. However, imbalances may be created by the interleaf friction of leaf-spring suspensions. Four-spring suspensions are particularly poor in this respect. However, interleaf friction is needed in order to provide necessary damping.

Typically, most tandem axles are very good at evenly distributing the weight on a tandem axle. Static measurements on tandem axles have shown that the largest variation is on the order of about 5 percent more weight on one axle than on the other.

4. <u>Dynamic Inter-Axle Load Transfer</u>: Dependent on coulomb friction and mechanisms intended to evenly distribute loads on both axles on a tandem set.

Inter-axle load transfer can occur in dynamic situations, such as braking or acceleration. Unfortunately, the mechanisms that are used to create a good static load equalization have just the opposite effect on dynamic load transfers. When a braking (or driving) force is applied on a tandem axle, there is often a load transfer between the axles of a tandem set. With air suspensions, the load transfer is usually large only if the linkages are different.

Inter-axle load transfers can be a particular problem because, during braking, the more lightly loaded axle will tend to lock up before the other. If the lockup occurs on the lead axle, then the directional stability is reduced. Directional stability can be completely lost if lockup occurs on the trailing axle.

Another unwanted result of poor load transfer is that the system can produce an under-damped mode. Occasionally, this can result in "tandem hop," which can cause a partial degradation of braking and handling performance.[9]

Four-spring suspensions tend to transfer the load to the trailing axle during braking more than other suspensions.

Dynamic inter-axle load transfer is measured in pounds of load transferred per pound of brake force. The transfer is positive in the direction of trailing to leading axle. A typical range of values is given in table 15.

Table 15. Typical range of inter-axle load transfer for truck suspension.[4]

Type of suspension	Range of inter-axle load transfer (lb/lb)
Air suspension	0.035 - (-0.018)
Four-spring	(-0.10) - (-0.185)
Walking beam	0.010 - (-0.030)

Note: 1 lb = 0.454 kg

5. Roll Center Height: Dependent on the line of action of the lateral suspension forces.

When a truck rolls (tilts sideways as when rounding a horizontal curve), it tends to roll about a specific point, called the roll center. The roll center is located at the line of action where the lateral forces interact between the chassis and the suspension. With a four-spring suspension, the leaf spring will determine the roll center location. Special links can be added to provide lateral forces on walking-beam and air suspensions which have an effect on the roll center height. Roll center heights are measured from the ground. Typical values are given in table 16.

Table 16. Typical range of roll center heights for truck suspensions.[4]

Type of suspension	Range of roll center height (in)
Front suspension	18.5 - 20
Air suspension	24 - 29.5
Four-spring	23 - 31
Walking beam	21.5 - 23
Single axle leaf spring	25 - 28

Note: 1 in = 2.54 cm

6. Roll Stiffness: Dependent on spring stiffness, lateral spacing, roll center height, and auxiliary mechanisms such as anti-sway bars.

Roll stiffness is a measure of a suspension system's resistance to rolling. As a truck rolls, the vertical springs deform to cause a resisting moment. This moment is dependent on the vertical spring constants and lateral spacing of the springs.

Leaf-spring suspensions do not need extra mechanisms for increased roll resistance. The steel leaf springs provide roll resistance since they must be bent along their length when roll occurs. Air-spring suspensions generally need some auxiliary mechanism to provide adequate roll resistance. Most air-spring systems use the axle housing to assist in roll resistance. This is done by rigidly clamping the axle housing to the trailing arms.[41] Very high roll stiffnesses can be achieved with this approach. Many trucks also have mechanisms, such as anti-sway bars, that resist rolling. Anti-sway bars are popular in Europe but are not widely used in the United States.

Leaf-spring suspensions use "slippers" at the ends to attach the springs. Due to the way these slippers are constructed, there is a small amount of freeplay which is present as the spring goes from compression to tension. This can create a lash effect that is significant in rolling situations. During a vehicle roll, the suspension should have an increase in resisting moment with an increase in roll angle. However, due to the freeplay, there is a point at which the suspension's resisting moment will remain constant as the roll of the truck body increases.[41] This happens only during a small part of the roll motion. For vehicles with high centers of gravity, this freeplay response can occur well below the rollover threshold.[41] On vehicles with low centers of gravity, the freeplay effect may not occur until the rollover threshold of the truck is approached or reached.

The height of the roll center plays an important part in the rolling tendency of a vehicle, as illustrated in figure 16. As a truck goes around a horizontal curve, the centrifugal force causes the truck body to roll about its roll center. This will also cause the center of gravity to produce a moment about the roll center, due to its shift in position. The higher the roll center (i.e., the closer it is to the center of gravity), the shorter the moment arm and the smaller the moment that is produced.

Ideally, the roll stiffness at each axle should be proportional to the weight on that axle, which means that the roll stiffness of the trailer axles should be about the same as that of the tractor's rear axles. However, this is not usually the case. More typically, the trailer has a harder suspension than the tractor.

As the truck rolls, the side to side weight distribution changes. The load transfer at each axle is proportional to the roll stiffness at that axle. The properties affected by the load transfer are stability, handling response time, roll steer, and rearward amplification.

The range of roll stiffnesses for the various suspensions has been measured with a load of 12,000 lb (5,500 kg) on the front axles and 16,000 lb (7,300 kg) on the rear axles. A typical range of roll stiffnesses on a per axle basis is given in table 17.

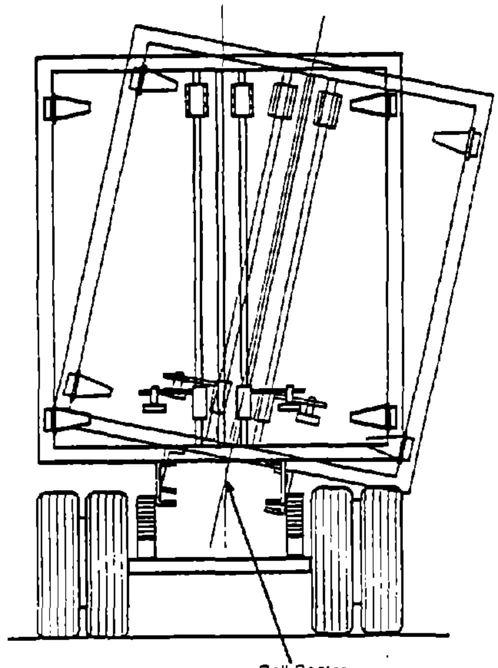


Figure 16. Diagram of roll by trailer body illustrating location of roll center.[4]

Table 17. Typical range of roll stiffness for truck suspensions.[4]

Type suspension	Range of roll stiffness (in-lb/deg)
Front suspension	0.017 - 0.025
Air suspension	0.025 - 0.090
Four-spring	0.065 - 0.140
Walking beam	0.070 - 0.160
Single-axle leaf spring	0.052 - 0.089

Note: 1 in = 2.54 cm
1 lb = 0.454 kg

7. Roll Steer Coefficient: Dependent on the layout of links that restrain the axles.

Nonsteering axles can deflect slightly to create a steering effect as a result of vehicle roll. As the truck body rolls, one side of the axle moves forward while the other side moves aft. This unintentional steering is created by the suspension and tire forces. The tendency to steer in a roll is measured with respect to the amount of vehicle roll present. This steering can greatly affect truck handling, particularly in a turn.

Most suspension systems are designed to control roll steer. However, some air suspensions lack special links to control roll steer and, as a result, the steering effect may be greater than is found with other types of suspensions.

The units used in measuring the roll steer coefficient are degrees of steer per degree of roll. A positive roll steer coefficient means that the axle will steer toward the outside of the turn; a negative coefficient means that the axle will steer toward the inside of the turn. A typical range of values is given in table 18 on a per-axle basis.

Table 18. Typical range of roll steer coefficients for truck suspensions.[4]

Type of suspension	Range of roll steer coefficient (deg/deg)
Air suspension	0.01 - 0.23
Four-spring	-0.04 - 0.23
Walking beam	0.16 - 0.21
Single axle leaf spring	0.0 - 0.07

8. Compliance Steer Coefficient: Dependent on tire aligning moments, and deflections of rubber bushings and other links.

Compliance steering is very similar to roll steering, except that it is caused by brake forces, side forces, and aligning moments. These forces and moments cause deflections of rubber bushings and other links, which in turn cause the steering effect. Compliance steering is not very large for non-steering axles.

Compliance steering is measured in degrees of steer per in-lb of applied moment. A typical range of values is given in table 19.

Table 19. Typical range of compliance steer coefficients for truck suspensions.[4]

Type of suspension	Range of compliance steering $(deg/in\text{-}lb)/10^6$
Air suspension	3.8 - 6.0
Four-spring	3.7 - 7.25
Walking beam	6.25 - 10.0
Single-axle leaf spring	4.9 - 5.4

Note: 1 in = 2.54 cm
1 lb = 0.454 kg

H. Rollover Threshold

A vehicle's resistance to rollover is measured by the maximum lateral acceleration that can be achieved without causing rollover. This maximum acceleration is known as the rollover threshold. Passenger cars generally have rollover thresholds of about 1.20 g.[42] The typical passenger car tracking a horizontal curve or making a turn at too high a speed is likely to skid off the road due to inadequate tire-pavement friction long before its rollover threshold is reached. Trucks, on the other hand, generally have rollover thresholds that are less than the available tire-pavement friction on dry pavements. Thus, rollovers are much more likely as a failure mode for trucks than for passenger cars.

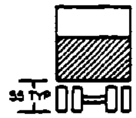
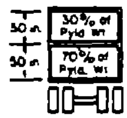
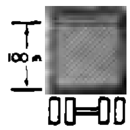
Truck rollovers are caused by high lateral acceleration in a turning maneuver. As lateral acceleration increases, the wheels on the inside of the turn begin to lift off the pavement. Generally, due to uneven load distribution and to uneven suspension, tire, and structural stiffness, all of the wheels will not begin to lift off the pavement at the same time. It is possible for one wheel of a truck to lift off the pavement without producing a rollover; however, this creates a very unstable situation that could ultimately lead to rollover.

Trucks have historically been considered to have rollover thresholds of 0.40 g or more. While this is undoubtedly true for most trucks, recent research has shown that trucks can have much lower rollover thresholds than previously suspected.[43,44] Figure 17 shows that certain loading configurations can produce truck rollover thresholds as low as 0.24 g.

As implied in figure 17, the rollover threshold of a truck is largely a function of its loading configuration. The following parameters of a truck's loading configuration affect its rollover threshold:

- Center of gravity (CG) height.
- Overall weight.
- Longitudinal weight distribution.
- Lateral weight distribution.

Trucks with higher centers of gravity have lower rollover thresholds. Thus, low rollover thresholds may be a particular problem for trucks carrying low-density freight, which tends to fill the trailer before reaching the maximum gross vehicle weight. Trucks carrying high-density freight that does not fill the entire trailer will generally have lower centers of gravity and higher rollover thresholds.

CASE		CONFIGURATION	WEIGHT (lbs) GVW	PAYLOAD CG HEIGHT (in)	ROLLOVER THRESHOLD (G's)
A.		Full Gross, Medium-Density Freight (34 lb/ft³)	80,000	83.5	.34
B.		"Typical" LTL Freight Load	73,000	95.0	.28
C.		Full Gross, Full Cube, Homogeneous Freight (18.7 lb/ft³)	80,000	105.0	.24
D.		Full Gross Gasoline Tanker	80,000	88.6	.32
E.		Cryogenic Tanker (H_e and H_2)	80,000	100.	.26

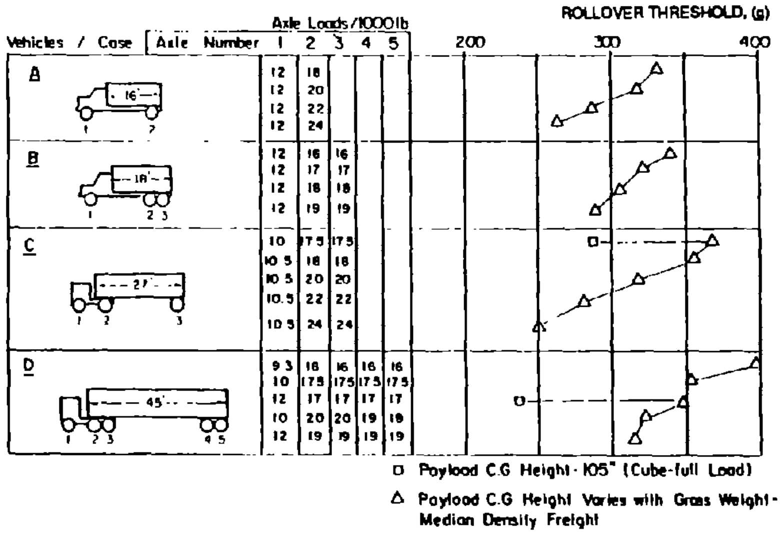
Note: 1 lb = 0.454 kg
1 in = 2.54 cm
1 ft = 0.305 m

Figure 17. Loading data and resulting rollover thresholds for example tractor-semitrailers at full load.[43]

The sensitivity of truck rollover threshold to these parameters is reviewed below based on results reported in a 1986 FHWA study, which have been confirmed by computer simulation analyses reported in appendix B of volume II.[44] These findings include:

- For the baseline case of an 80,000-lb (36,400-kg) single-semitrailer truck, with medium density cargo, loaded evenly left to right and fore and aft, on a 96-in (244-cm) trailer, the computer rollover threshold is 0.35 g.

- Figure 18 illustrates some typical variations in rollover threshold with axle loadings for four different trucks: a two-axle single-unit truck; a three-axle single-unit truck; a short, three-axle single-semitrailer truck (equivalent to the first trailer of a pre-STAA twin-trailer combination); and a five-axle tractor semitrailer truck (equivalent to a long, pre-STAA single-semitrailer truck).



Note: 1 lb = 0.454 kg
1 in = 2.54 cm

Figure 18. Influence of axle load variations on rollover threshold.[44]

- The distribution of weight between axles (fore and aft) has a minimal effect on rollover threshold. For a given gross vehicle weight, shifting weight forward onto the tractor steering axle is the worst case, but the reduction in rollover threshold is only about 0.01 g.

- Adding weight to the truck by adding more of the same density cargo on top of the existing load raises the CG and lowers the rollover threshold. The effect is a reduction of about 0.01 g per added ton (0.01 g per added Mg), so increasing the weight to 88,000 lb (40,000 kg) would reduce the rollover threshold by about 0.04 g.

- Four vehicle width parameters influence the rollover threshold: width of trailer body, width between trailer tires, width between spring centers, and tractor width. Rollover threshold decreases as each of these widths increases. Of these four width parameters, the tractor width has the largest effect on rollover threshold. However, this is of little practical significance, because the change from 96- to 102-in (244- to 259-cm) trailers has been made without using wider tractors. However, if the other three width parameters are widened by amounts typical of the differences between 96- and 102-in (244- and 259-cm) trucks, the rollover threshold is increased by about 0.03 g.

  There is some concern about whether 102-in (259-cm) width trailers are being placed on chassis with 96-in (244-cm) wheels. This initially appeared to be only a transitional problem that would disappear after most manufacturers have completely changed over to 102-in (259-cm) trailer production. However, there is now concern that some 102-in (259-cm) trailers will continue to be placed on 96-in (244-cm) wheels, because the railroads are unwilling to modify piggyback equipment to accept trucks with 102-in (259-cm) wheels. Data for 1988 from the Truck Trailer Manufacturers Association show that 71 percent of van trailers for highway use and 96 percent of trailer-on-flat-car (TOFC) units are being produced with 102-in (259-cm) widths. TOFC units appear to constitute about 9.5 percent of van trailer production.[9]

- Retaining the same gross vehicle weight (at 80,000 lb or 36,400 kg), but assuming less dense cargo, would raise the CG height and lower the rollover threshold. The rollover threshold is reduced by about 0.005 g for every inch the CG is raised. For the baseline vehicle, the payload CG is at a height of 83.5 in and the composite truck CG is at 80 in. A "typical," fully loaded single semitrailer truck used by a less-than-truckload (LTL) carrier has a payload CG height of 95 in (241 cm), and the worst case scenario of a "cubed out" truck at 80,000 lb (36,400 kg) has a CG height of 105 in (267 cm). The rollover thresholds of these three trucks are 0.35, 0.28, and 0.24 g, respectively.

- If the load is not centered left to right in the truck, its rollover threshold is raised on turns in the direction to which the load is offset, and reduced in turns in the opposite direction. The effect can be quite large--about 10 percent for each 3 in (7.6 cm) of offset. An offset of 3 in (7.6 cm) would correspond to an (incorrect) loading of 96 in (244 cm) pallets along one side of a 102-in (259 cm) trailer. This would result in a lowering of the rollover threshold by 0.03 g for either left or right turns, and an increase of rollover threshold by 0.03 g for the opposite turn.

- For the same width, weight, and CG height, double-trailer trucks consistently have rollover thresholds 0.03 to 0.05 g higher than semis. Thus, semis are the vehicles of most concern.

  Some concern remains about rearward amplification in doubles in sudden maneuvers, such as obstacle avoidance. Rearward amplification can lead to rollover of the rear trailer. However, this is more of a concern in emergency maneuvers than in normal tracking of a curve or turn, which is the basis for geometric design.

- Trailer length has no direct effect on the truck rollover threshold except through its influence on the amount of load carried, the loading pattern, and, consequently, the CG height. A longer trailer would typically have a higher rollover threshold than a shorter trailer carrying the same type and amount of cargo because the payload CG would be lower. If the amount of cargo (of the same density) were increased in proportion to the increase in trailer length and the trailer were uniformly loaded, the rollover threshold would remain about the same. Thus, increasing the trailer length does not lower the rollover threshold; in many cases, the opposite is true.

- All of the rollover thresholds given above were developed from simulation models. Several experiments have been performed to compare the model results to real-world observations. The actual experimental rollover thresholds were consistently about 0.03 g higher than predicted by the model.

- A 1986 FHWA study analyzed BMCS accident data representing 9,000 single-vehicle accidents involving 5-axle semis.[44] Of these, over 2,000 resulted in a rollover. Using the reported gross vehicle weight, and assuming medium-density freight and a 96-in (244-cm) width, the rollover thresholds were calculated and the distribution shown in figure 19 was plotted. The lowest calculated rollover threshold was about 0.39 g. If this were adjusted to the worst-case scenario of a light-density cargo, with cubed-out loading, a rollover threshold of 0.27 to 0.28 g would result.

- A rollover threshold of about 0.30 g appears to be appropriate for design. The worst-case rollover threshold of 0.24 g should be adjusted upwards by 0.03 g to account for the wider trailers now in use and by 0.03 g to account for the differences between experimental and model results. This compares well with the conclusions of the accident study cited above, if one also adds 0.03 g to those results to account for wider trailers.

  Further adjustments could be made. One could deduct 0.03 g from the rollover threshold to account for offset loads, but this would not be in agreement with real-world accident data. However, the rollover thresholds cited above address the worst case, while commonly accepted design practice is to design for something less extreme than the worst case (e.g., the 85th percentile). It is probable that the 85th percentile of rollover threshold for loaded trucks is more like 0.40 g. Thus, it appears that design of horizontal curves based on a rollover threshold of 0.30 g would be very conservative.

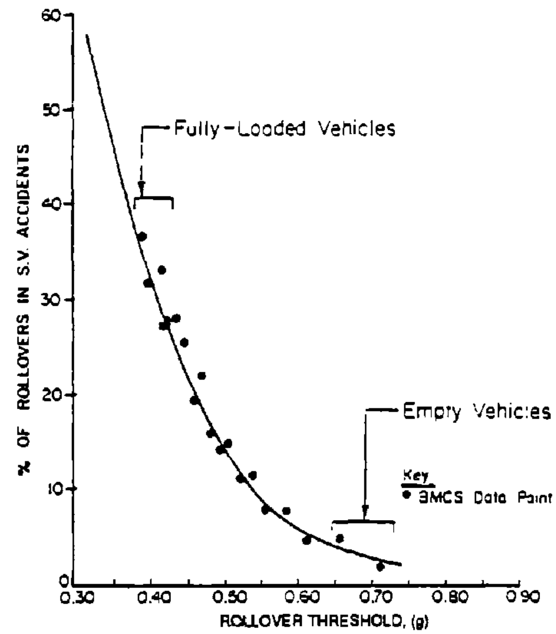


Figure 19. Percent of single-truck accidents in which rollover occurs as a function of rollover threshold.[44]

III. HIGHWAY DESIGN AND OPERATIONAL CRITERIA

This section provides a preliminary review of the adequacy of individual highway design and operational criteria to accommodate trucks. The review includes each of the highway design and operational criteria identified in table 1 as being wholly or partly based on a vehicle characteristic. Speed-change lane criteria for trucks have been omitted from this report because they are being addressed in a current NCHRP study.[45] Highway capacity criteria have been omitted because truck effects are addressed thoroughly in the 1985 Highway Capacity Manual.[46]

The review of each individual highway design and operational criterion includes a discussion of the criterion currently used by highway agencies, typically based on the AASHTO Green Book or the MUTCD; a critique of that criterion based on recent research concerning truck characteristics or concerning the traffic operational and safety effects of the criterion; a sensitivity analysis of the effects of changing the vehicle characteristics used to base the criterion on truck characteristics rather than passenger car characteristics; and recommendations concerning the need to revise existing highway design and operational criterion to accommodate trucks. Changes in existing highway design and operational criteria are recommended only where they appear to be cost effective.

Each highway design and operational criterion is discussed below.

A. Stopping Sight Distance

1. Current Design and Operational Criteria

Sight distance is the length of roadway ahead that is visible to the driver. The minimum sight distance available on the roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. This minimum sight distance, known as stopping sight distance, is the basis for design criteria for crest vertical curve length and minimum offsets to horizontal sight obstructions. Not only is the provision of stopping sight distance critical at every point on the roadway, but stopping sight distance also forms the basis for a number of additional highway design and operational criteria, including intersection sight distance, railroad-highway grade crossing sight distance, and advance warning sign placement criteria.

a. Stopping Sight Distance Criteria

Stopping sight distance is determined as the summation of two terms: brake reaction distance and braking distance. The brake reaction distance is the distance travelled by the vehicle from when the driver first sights an object necessitating a stop to the instant the brakes are applied. The braking distance is the distance required to bring the vehicle to a stop once the brakes are applied.

The stopping sight distance criteria in the AASHTO Green Book are based on the following equation:

$$S = 1.47\ t_{pr}V + \frac{V^2}{30f} \tag{25}$$

where: S = Stopping sight distance (ft)

t_{pr} = Perception-reaction time (s)

V = Initial vehicle speed (mi/h)

f = Coefficient of tire-pavement friction

The first term of Equation (25) represents the brake reaction distance, while the second term represents the braking distance. The factors that influence braking distances are discussed extensively in section II-A and appendix A of this report. The coefficient of sliding friction is used by AASHTO in equation (25) to determine the braking distance for a locked wheel stop by a passenger car.

Table 20 presents the AASHTO Green Book criteria for stopping sight distance. These criteria are based on an assumed perception-reaction time (t_{pr}) of 2.5 s and the assumed values of speed and coefficient of friction shown in the table. The two values shown in the table for the assumed speed, brake reaction distance, braking distance on level, and stopping sight distance represent minimum and desirable designs, respectively. The subsequent analyses in this report are based on the desirable sight distances, which are applicable to stopping by a vehicle traveling at the design speed of the highway.

b. Correction of Stopping Sight Distance Criteria for Grades

Stopping sight distance is also affected by roadway grade because longer braking distance is required on a downgrade and shorter braking distance is required on an upgrade. The Green Book criteria for grade effects on stopping sight distance are derived with the following equation:

$$S = 1.47\ t_{pr}V + \frac{V^2}{30(f+G)} \tag{26}$$

where: G = percent grade/100 (+ for upgrade, - for downgrade)

Table 20. AASHTO criteria for stopping sight distance.[1]

Design Speed (mph)	Assumed Speed for Condition (mph)	Brake Reaction Time (sec)	Brake Reaction Distance (ft)	Coefficient of Friction f	Braking Distance on Level (ft)	Stopping Sight Distance Computed[a] (ft)	Stopping Sight Distance Rounded for Design (ft)
20	20-20	2.5	73.3- 73.3	0.40	33.3- 33.3	106.7-106.7	125-125
25	24-25	2.5	88.0- 91.7	0.38	50.5- 54.8	138.5-146.5	150-150
30	28-30	2.5	102.7-110.0	0.35	74.7- 85.7	177.3-195.7	200-200
35	32-35	2.5	117.3-128.3	0.34	100.4-120.1	217.7-248.4	225-250
40	36-40	2.5	132.0-146.7	0.32	135.0-166.7	267.0-313.3	275-325
45	40-45	2.5	146.7-165.0	0.31	172.0-217.7	318.7-382.7	325-400
50	44-50	2.5	161.3-183.3	0.30	215.1-277.8	376.4-461.1	400-475
55	48-55	2.5	176.0-201.7	0.30	256.0-336.1	432.0-537.8	450-550
60	52-60	2.5	190.7-220.0	0.29	310.8-413.8	501.5-633.8	525-650
65	55-65	2.5	201.7-238.3	0.29	347.7-485.6	549.4-724.0	550-725
70	58-70	2.5	212.7-256.7	0.28	400.5-583.3	613.1-840.0	625-850

Note: 1 mi = 1.61 km
1 ft = 0.305 m

c. Application of Stopping Sight Distance Criteria to Crest Vertical Curves

Vertical crests limit the sight distance of the driver. Crest vertical curves designed in accordance with the AASHTO Green Book criteria should provide stopping sight distance at least equal to the requirements of table 20 at all points along the curve. The minimum length of a crest vertical curve as a function of stopping sight distance (S) is calculated by AASHTO as:

For S less than L_{min}:

$$L_{min} = \frac{AS^2}{100\left(\sqrt{2H_e} + \sqrt{2H_o}\right)^2} \quad (27)$$

For S greater than L_{min}:

$$L_{min} = 2S - \frac{200\left(\sqrt{H_e} + \sqrt{H_o}\right)^2}{A} \quad (28)$$

where: L_{min} = Minimum length of vertical curve (ft)

S = Stopping sight distance (ft)

A = Algebraic difference in percent grade

H_e = Height of driver's eye above roadway surface (ft)

H_o = Height of object above roadway surface (ft)

Equations (27) and (28) are based on the mathematical properties of a parabolic curve. The AASHTO Green Book suggests that it is typical practice to use a minimum vertical curve length that is at least three times the value of the design speed (expressed in mi/h). For stopping sight distance, the driver eye height (H_e) used by AASHTO is 42 in (107 cm) and the object height (H_o) used is 6 in (15 cm). Table 21 presents the minimum vertical curve lengths to attain the desirable stopping sight distance criteria in table 20 as a function of design speed.

Table 21. Minimum vertical curve lengths (ft) needed to provide AASHTO stopping sight distance.

Algebraic difference in percent grade	Design speed (mi/h) 20	30	40	50	60	70
2	60	90	150	260	610	1,070
4	60	120	300	650	1,220	2,130
6	60	170	450	970	1,820	3,190
8	70	240	600	1,280	2,420	4,260
10	90	290	740	1,610	3,030	5,320

Note: Based on AASHTO driver eye height of 42 in (107 cm) for a passenger car.
1 mi = 1.61 km

d. Application of Stopping Sight Distance Criteria to Horizontal Curves

Sight distance can also be limited by obstructions on the inside of horizontal curves, such as trees, buildings, retaining walls, and embankments. Horizontal curves designed in accordance with the AASHTO Green Book should provide sight distance at least equal to the requirements of table 20 along the entire length of the curve. For a circular horizontal curve, the line of sight is a chord of that curve and the sight distance is measured along the centerline of the inside lane. The minimum offset to a horizontal sight obstruction at the center of the curve (known as the middle ordinate of the curve) is computed in accordance with the following equation:

$$M = R\left(1 - \cos \frac{28.65\ S}{R}\right) \tag{29}$$

where: M = middle ordinate of curve (ft)

R = radius of curve (ft)

2. Critique of Design and Operational Criteria

This section reviews the recent literature relevant to stopping sight distance criteria and their application to crest vertical curves and horizontal curves. The criteria are based on consideration of the passenger car as the design vehicle. The critique calls attention to differences between passenger cars and trucks that are relevant to stopping sight distance design.

Table 22 summarizes the historical evolution of the AASHTO stopping sight distance criteria. This summary addresses the following aspects of stopping sight distance criteria:

- Assumed speed for design.
- Brake reaction time.
- Coefficient of tire-pavement friction.
- Eye height.
- Object height.

Each of these factors is discussed below.

a. Assumed Speed for Design

The assumed speed for stopping sight distance design purposes has historically been less than the design speed of the highway based on the assumption that drivers travel more slowly on wet pavements than on dry pavements. This assumption was used to derive the lower values of stopping sight distance in table 20. AASHTO notes that recent data have shown that drivers travel about as fast on wet pavements as they do on dry pavements. Therefore, the higher values of stopping sight distance in table 20 are based on braking by a vehicle traveling at the design speed of the highway. All analyses of stopping sight distance in this study have been conducted with the assumption that the braking vehicle, passenger car or truck, is initially traveling at the design speed of the highway.

b. Brake Reaction Time

The AASHTO criteria for stopping sight distance are based on a brake reaction time of 2.5 s. This choice for brake reaction time has been confirmed as appropriate for most drivers in several studies.[12,47] This value appears well supported and has not been varied in this study.

The brake reaction time is a driver characteristic and is assumed to be applicable to truck drivers as well as passenger car drivers. In fact, experienced professional truck drivers could reasonably be expected to have shorter brake reaction times than the driver population as a whole. On the other hand, the air brake systems commonly used in tractor trailer combination trucks have an inherent delay of approximately 0.5 s in brake application.[4] For purpose of this analysis, it is assumed that the factors offset one another and that the 2.5-s brake reaction time is appropriate for trucks.

Table 22. Evolution of AASHTO stopping sight distance policy.[48]

Year	Eye height (in)	Object height (in)	Perception/ reaction time (s)	Assumed coefficient of tire-pavement friction	Effective assumed speed for design	change from previous policy
1940 (49)	54	4	Variable-- 3.0 s @ 30 mi/h to 2.0 s @ 70 mi/h	DRY-- f Ranges from 0.50 @ 30 mi/h to 0.40 @ 70 mi/h	DESIGN SPEED	--
1954 (50)	54	4	2.5	WET-- f Ranges from 0.36 @ 30 mi/h to 0.29 @ 70 mi/h	Lower than design speed (28 mi/h @ 30 mi/h design speed; 59 mi/h @ 70 mi/h design speed	No net change in design distances
1965 (51)	45	6	2.5	WET-- f Ranges from 0.36 @ 30 mi/h to 0.27 @ 80 mi/h	Lower than design speed (28 mi/h @ 30 mi/h design speed; 64 mi/h @ 80 mi/h design speed	No net change in design distances
1971 (52)	45	6	2.5	WET-- f Ranges from 0.35 @ 30 mi/h to 0.27 @ 80 mi/h	Minimum Values-- Same as 1965; Desirable Values-- DESIGN SPEED	Desirable values are up to 250 ft greater than minimum value
1984 (1)	42	6	2.5	WET-- f slightly lower than 1970 values for higher speeds	Minimum Values-- Same as 1965; Desirable Values-- DESIGN SPEED 1970	Computed values always rounded up giving slightly higher value than

Note: 1 in = 2.54 cm
1 mi = 1.61 km

c. Coefficient of Tire-Pavement Friction

The coefficients of friction shown in table 20 were chosen from the results of several studies cited in figure III-1 of the AASHTO Green Book and are intended to represent the deceleration rates used by a passenger car in locked-wheel braking on a poor, wet pavement. The results cited in the AASHTO Green Book that most closely match the criteria in table 20 come from a 1951 study by and are based on locked-wheel skid test results obtained for new passenger car tires.[53]

A critical fact concerning truck stopping distance is that trucks cannot safely make a locked-wheel stop without the risk of losing control of the vehicle. The discussion of braking distances in Section II-B of this report has shown that the deceleration rates used by trucks in making controlled stops are generally lower than the deceleration rates used by passenger cars making locked-wheel stops. The estimates of deceleration rate and braking distances shown in table 6 have been utilized in sensitivity analyses of stopping sight distance requirement for trucks later in this section.

d. Driver Eye Height

The minimum crest vertical curve criteria for stopping sight distance in table 21 are based on a driver eye height for passenger cars of 42 in (107 cm). The driver eye heights for trucks are much higher than for passenger cars, which may partially or completely offset their longer braking distances on crest vertical curves. However, the higher eye heights of truck drivers provide no comparable advantage at sight obstructions on horizontal curves unless the truck driver is able to see over the obstruction.

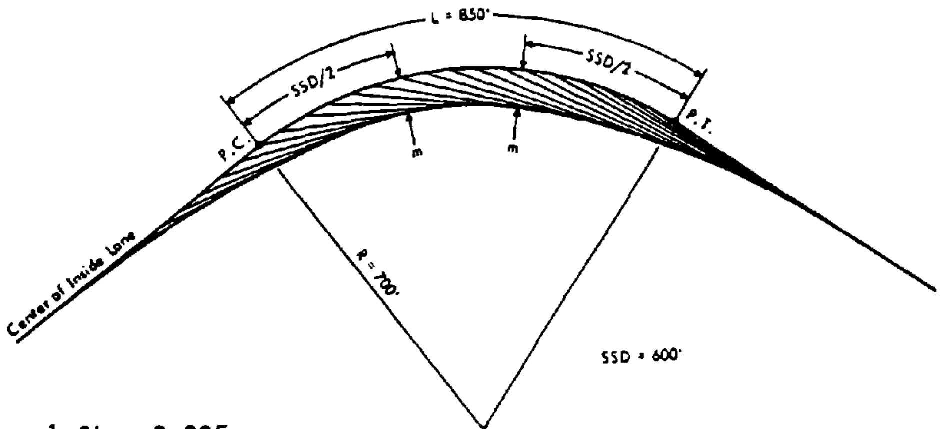
The review of truck driver eye heights in section II-C concluded that truck driver eye height can range from 71.5 to 112.5 in (182 to 286 cm). A 1983 FHWA study estimated the average driver eye height for a conventional tractor to be 93 in (236 cm).[22] This value was also used in the studies of stopping sight distance in NCHRP Report 270. However, because some driver eye heights may be very much lower than 93 in (236 cm), sensitivity analyses in this study have been conducted for values of both 75 and 93 in (190 and 236 cm).

e. Object Height

The object height used in determining the crest vertical curve lengths in table 21 is 6 in (15 cm). As shown in table 22, a 4-in (10 cm) object height was used prior to 1965. The AASHTO Green Book presents the object height as an arbitrary rationalization of possible hazardous objects that could be found in the roadway. Others maintain that, historically, the object height represented a subjective tradeoff of the cost of providing sight distance to the pavement and did not represent any particular hazard.[48] The recent analysis of this issue in NCHRP Report 270 assumed that the object height was meant to represent a specific possible hazard, but questioned the use of a 6-in (15 cm) object based on another recent study which found that about 30 percent of compact and subcompact passenger cars could not clear an object of that height.[12,54] Whatever interpretation of object height is chosen the crest vertical lengths for trucks should not be affected, since trucks typically have underclearances substantially greater than 6 in (15 cm).

f. Horizontal Sight Obstructions

Increased eye height provides truck drivers no advantage over passenger car drivers at a horizontal sight obstruction, unless the truck driver is able to see over the obstruction. However, NCHRP Report 270 indicates that the minimum offset to a horizontal sight obstruction (represented by the middle ordinate of the curve computed with equation (29)) is normally required only near the center of a horizontal curve.[12] Figure 20 illustrates a sight distance envelope or "clear sight zone" within which horizontal sight obstructions should not be present. The figure illustrates that less offset to horizontal sight obstructions is required within a distance to the ends of the curve equal to half the stopping sight distance.



Note: 1 ft = 0.305 m

Figure 20. Example sight obstruction envelope on horizontal curves for condition where the stopping sight distance is less than the length of the curve.

Another problem associated with stopping sight distance on horizontal curves is that the tire-pavement friction available for braking is reduced by the portion of the available tire-pavement friction that is required for cornering.[12,55] NCHRP Report 270 expresses the available friction for braking on a horizontal curve as:[12]

$$f^2 = f_t^2 - \left(\frac{V^2}{15R} - e\right)^2 \tag{30}$$

where: f = Coefficient of friction available for braking

f_t = Total available coefficient of friction

V = Vehicle speed (mi/h)

R = Radius of curvature (ft)

e = Superelevation rate (ft/ft)

Equation (30) implies that the required stopping sight distances on horizontal curves should be longer than on tangents.

3. Sensitivity Analysis

A sensitivity analysis was performed to investigate the differences in stopping sight distance requirements for trucks and passenger cars. The stopping sight distance criteria for passenger cars were represented by the AASHTO criteria. The sensitivity analysis also examined the implications of the stopping sight distance analysis results for crest vertical curves and for horizontal sight obstructions.

a. Stopping Sight Distance

Stopping sight distance criteria for trucks were derived using the AASHTO stopping sight distance relationship given in equation (25). The stopping sight distance criteria for trucks were based on the same brake reaction time (t_{pr}) as the AASHTO criteria. The design speed of the highway is used as the initial vehicle speed in the braking maneuver. Three cases are considered for the coefficients of friction or deceleration rates used by truck drivers for controlled stops. These are a truck with a conventional braking system and the worst-performance driver, a truck with a conventional braking system and the best-performance driver, and a truck with an antilock brake system. The estimated deceleration rates for these three cases, shown in table 6, are based on braking by an empty tractor-semitrailer truck, with good tires, on a poor, wet road.

Table 23 presents the stopping sight distance requirements for trucks derived from the data discussed above, in comparison to the current AASHTO criteria.

The sensitivity analysis based on the use of the current AASHTO stopping sight distance model forms the basis for determining truck requirements. However, this model is in need of a thorough review to determine if it truly meets the sight distance needs of drivers.

Table 23. Stopping sight distance requirements for trucks in comparison to current AASHTO criteria.

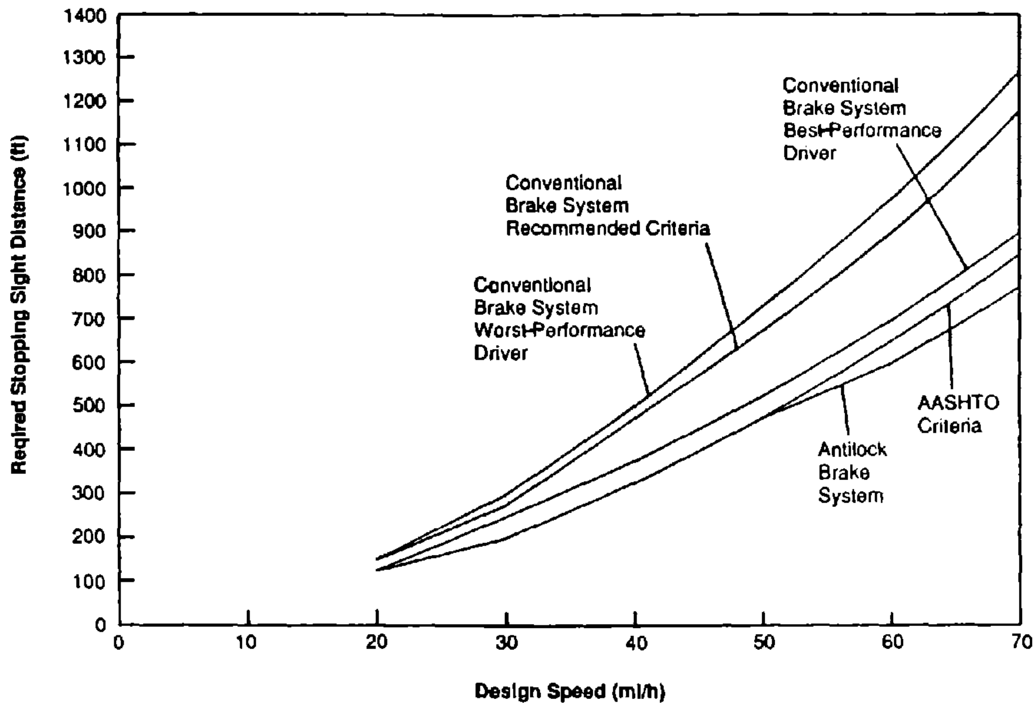
Design speed (mi/h)	Required stopping sight distance (ft)			
		Controlled braking[a]		
	AASHTO criteria	Worst-performance driver	Best-performance driver	Antilock brake system
20	125	150	125	125
30	200	300	250	200
40	325	500	375	325
50	475	725	525	475
60	650	975	700	600
70	850	1,275	900	775

[a] Based on deceleration rates and braking distances presented in table 6.

Note: 1 mi =1.61 km
1 ft = 0.305 m

Table 23 and figure 21 show that the worst performance driver with a conventional braking system requires substantially more stopping sight distance than the AASHTO criteria, up to 425 ft (130 m) more sight distance for a 70 mi/h (113 km/h) design speed. On the other hand, the stopping sight distance requirements for the best-performance driver with a conventional braking system are only slightly higher than the current AASHTO criteria. Thus, the assumption made about the braking performance capability, or braking control efficiency, of the driver is critical to stopping sight distance. There are essentially no data available to indicate the actual distribution of braking control efficiencies for on-the-road truck drivers.

Table 23 shows three potential sets of stopping sight distance criteria for trucks, each of which differ to some extent from current AASHTO criteria. If antilock brake systems for trucks do not come into use, a choice of stopping sight distance criteria must be made in the range from between the worst performance driver (62 percent driver control efficiency) and the best-performance driver (100 percent driver control efficiency). It would not be fair to select either end of this range as the basis for stopping sight distance design. Although the literature indicates that, with practice at a test track, truck drivers can quickly learn to make emergency stops at nearly 100 percent driver control efficiency, real-world experience and opportunities for practice are rare. Current truck driver training programs do not generally include practice in making emergency stops. On the other hand, use of 62 percent control efficiency in design would be unnecessarily conservative.



Note: 1 ft = 0.305 m
1 mi = 1.61 km

Figure 21. Comparison of stopping sight distance requirements for trucks to current AASHTO criteria.

Highway design is seldom based on extreme values of design parameters, but is more typically based on specific percentiles of the parameter distribution (e.g., the 85th percentile). Since the real-world distribution of truck driver braking performance is unknown, the design value of 70 percent driver control efficiency was selected based on engineering judgment. Table 24 presents candidate stopping sight distance criteria for trucks, based on 70 percent driver control efficiency, in comparison to the current AASHTO criteria. It is evident from table 23 that, if antilock brake systems do come into fairly universal use and achieve the performance projected in table 6, the current AASHTO stopping sight distance criteria should be adequate for trucks, and the candidate revision presented in table 24 would not be necessary.

Figure 21 compares the stopping sight distance requirements for passenger cars and trucks given in table 23 and the candidate criteria for trucks in table 24.

Table 24. Candidate stopping sight distance criteria for trucks.

Design speed (mi/h)	Stopping sight distance (ft)	
	AASHTO criteria	Candidate criteria for trucks[a]
20	125	150
30	200	275
40	325	475
50	475	675
60	650	900
70	850	1,175

[a] Not applicable if antilock brake systems for trucks come into nearly universal use.

Note: 1 mi = 1.61 km
1 ft = 0.305 m

b. Crest Vertical Curve Lengths

Table 25 shows the minimum vertical curve lengths for a range of design speeds and algebraic differences in grade based on the stopping sight distance requirements for trucks in tables 23 and 24. The comparable AASHTO criteria for passenger cars are presented in table 21. All of the vertical curve lengths in table 25 are based on a 6-in (15 cm) object height. The AASHTO criteria are based on a 42-in (107 cm) driver eye height and the truck criteria are based on driver eye heights of 75 and 93 in (190 and 236 cm).

Table 25. Minimum vertical curve lengths (ft) to provide stopping sight distance for trucks.

Algebraic difference in percent grade	Design speed (mi/h) 20	30	40	50	60	70
			TRUCK (driver eye height = 75 in)			
Conventional Brake System with 70% Driver Control Efficiency[a]						
2	60	90	200	300	740	1,300
4	60	150	390	850	1,510	2,600
6	60	210	600	1,270	2,260	3,890
8	80	300	800	1,700	3,020	5,190
10	80	370	1,000	2,120	3,770	6,490
Conventional Brake System with Best Performance Driver[b]						
2	60	90	130	260	340	750
4	60	100	210	520	910	1,530
6	60	110	380	780	1,360	2,300
8	60	200	450	1,040	1,810	3,050
10	80	250	630	1,300	2,260	3,820
Antilock Brake System[b]						
2	60	90	120	200	350	510
4	60	90	130	400	700	1,150
6	60	120	300	600	1,040	1,720
8	60	140	400	800	1,400	2,300
10	60	200	500	1,000	1,730	2,870
			TRUCK (driver eye height = 93 in)			
Conventional Brake System with 70% Driver Control Efficiency[a]						
2	60	90	170	360	550	1,100
4	60	130	300	720	1,270	2,190
6	60	150	510	1,080	1,910	3,290
8	70	250	670	1,430	2,550	4,380
10	90	310	840	1,790	3,180	5,470
Conventional Brake System with Best Performance Driver[b]						
2	60	90	120	220	390	560
4	60	90	220	430	770	1,290
6	60	130	320	660	1,150	1,930
8	60	150	430	880	1,530	2,580
10	60	210	540	1,080	1,910	3,220
Antilock Brake System[b]						
2	60	90	120	190	320	390
4	60	90	190	340	640	1,060
6	60	110	260	560	960	1,590
8	60	120	370	740	1,270	2,120
10	60	180	460	920	1,590	2,650

[a] Based on stopping sight distances shown in table 24.
[b] Based on stopping sight distances shown in table 23.
Note: 1 ft = 0.305 m; 1 mi = 1.61 km; 1 in = 2.54 cm

The data in table 25 indicate that the minimum vertical curve lengths for the candidate stopping sight distance criteria for trucks are always longer--in some cases, by a substantial margin--than current AASHTO criteria. On the other hand, the minimum vertical curve lengths for a truck with an antilock brake system or for the best performance driver in a truck with a conventional brake system are virtually always shorter than the current AASHTO criteria. Stated another way, both the truck with the antilock brake system and the best performing driver with a conventional brake system will always have enough stopping sight distance on a curve designed in accordance with AASHTO criteria.

Finally, the data in table 25 show that the minimum vertical curve lengths are not very sensitive to the difference between 75 and 93 in in driver eye height. The maximum difference in vertical curve lengths between these minimum and average driver eye heights is approximately 1,000 ft (300 m) in one extreme case, while most of the differences are substantially shorter.

c. Horizontal Sight Obstructions

The differences in stopping sight distance between passenger cars and trucks shown in tables 23 and 24 are generally not mitigated by increased driver eye height as in the case of vertical sight restrictions. In fact, as shown in equation (30) the sight distance criteria for horizontal curves should actually be somewhat higher, as a function of curve radius and superelevation, than for tangents.

4. Recommended Revisions to Design and Operational Criteria

The sensitivity analysis presented above has shown that current AASHTO criteria for stopping sight distance may not be adequate to accommodate trucks unless antilock brake systems for trucks are required by government regulation or come into nearly universal use. Table 24 presents a candidate table 25 revision to stopping sight distance criteria that would be adequate for trucks with conventional brake systems.

A cost effectiveness analysis was performed to determine whether adoption of the candidate stopping sight distance criteria could reasonably be expected to provide sufficient safety benefits to justify the additional highway construction costs for improved sight distance. Appendix F in volume II documents the methodology used for this cost effectiveness analysis and the stopping sight distance analyses are presented in that appendix as examples. Tables 26 and 27 summarize the results of those analyses for rural two-lane highways and rural freeways, respectively. Each table presents the minimum percent reduction in truck accidents on crest vertical curves that would be required to make an improvement of sight distance to the candidate criteria in Table 24 cost effective. The percentage reductions apply to all truck accidents located at any point on a vertical curve, not just to truck accidents that are near the actual crest or that have causes related to inadequate sight distance.

Table 26. Minimum percent reduction in truck accidents on crest vertical curves required for cost effectiveness of improved stopping sight distance on rural two-lane highways.

Average daily traffic volume (veh/day)	Percent trucks				
	1%	5%	10%	20%	30%
Scenario 1 -- New Construction or Major Reconstruction					
1,000	629.1	125.8	62.9	31.5	21.0
2,000	359.5	71.9	36.0	18.0	12.0
3,000	251.7	50.3	25.2	12.6	8.4
4,000	193.6	38.7	19.4	9.7	6.5
5,000	157.3	31.5	15.7	7.9	5.2
6,000	132.5	26.5	13.3	6.6	4.4
7,000	114.4	22.9	11.4	5.7	3.8
8,000	100.7	20.1	10.1	5.0	3.4
9,000	89.9	18.0	9.0	4.5	3.0
10,000	81.2	16.2	8.1	4.1	2.7
11,000	74.0	14.8	7.4	3.7	2.5
12,000	68.0	13.6	6.8	3.4	2.3
13,000	62.9	12.6	6.3	3.2	2.1
14,000	58.5	11.7	5.9	2.9	2.0
15,000	54.7	10.9	5.5	2.7	1.8
Scenario 2 -- Rehabilitation					
1,000	7,801.4	1,560.3	780.1	390.1	260.1
2,000	4,458.0	891.6	445.8	222.9	148.6
3,000	3,120.6	624.1	312.1	156.0	104.0
4,000	2,400.4	480.1	240.0	120.0	80.0
5,000	1,950.4	390.1	195.0	97.5	65.0
6,000	1,642.4	328.5	164.2	82.1	54.8
7,000	1,418.4	283.7	141.8	70.9	47.3
8,000	1,248.2	249.7	124.8	62.4	41.6
9,000	1,114.5	222.9	111.5	55.8	37.2
10,000	1,006.6	201.3	100.7	50.3	33.6
11,000	917.8	183.6	91.8	45.9	30.6
12,000	843.4	168.7	84.3	42.2	28.1
13,000	780.1	156.0	78.0	39.0	26.0
14,000	725.7	145.1	72.6	36.3	24.2
15,000	678.4	135.7	67.8	33.9	22.6

Table 27. Minimum percent reduction in truck accidents on crest vertical curves required for cost effectiveness of improved stopping sight distance on rural freeways.

Average daily traffic volume (veh/day)	Percent trucks 1%	5%	10%	20%	30%
Scenario 1 -- New Construction or Major Reconstruction					
2,000	1,028.6	205.7	102.9	51.4	34.3
3,000	806.2	161.2	80.6	40.3	26.9
4,000	662.9	132.6	66.3	33.1	22.1
5,000	562.8	112.6	56.3	28.1	18.8
10,000	320.7	64.2	32.1	16.0	10.7
15,000	224.3	44.9	22.4	11.2	7.5
20,000	245.2	49.0	24.5	12.3	8.2
25,000	187.5	37.5	18.7	9.4	6.3
30,000	149.6	29.9	15.0	7.5	5.0
35,000	123.0	24.6	12.3	6.2	4.1
40,000	103.4	20.7	10.3	5.2	3.5
45,000	88.4	17.7	8.8	4.4	3.0
50,000	76.7	15.3	7.7	3.8	2.6
Scenario 2 -- Rehabilitation					
2,000	19,436.8	3,887.3	1,943.7	971.8	648.9
3,000	15,234.1	3,046.8	1,523.4	761.7	507.8
4,000	12,525.8	2,505.2	1,252.6	626.3	418.5
5,000	10,635.1	2,127.0	1,063.5	531.8	354.5
10,000	6,060.9	1,212.2	606.1	303.1	202.0
15,000	4,238.1	847.6	423.8	211.9	141.3
20,000	4,633.7	926.7	463.4	231.7	154.5
25,000	3,542.2	708.4	354.2	177.1	118.1
30,000	2,826.2	565.2	282.6	141.3	94.2
35,000	2,323.6	464.7	232.4	116.2	77.5
40,000	1,953.4	390.7	195.3	97.7	65.1
45,000	1,670.8	334.2	167.1	83.5	55.7
50,000	1,449.1	289.1	144.9	72.4	48.3

Two costing scenarios for sight distance improvements are addressed in each table. Scenario 1 applies to new construction or major reconstruction in which the only expenditure required to improve the stopping sight distance is additional earthwork. Scenario 2 applies to rehabilitation projects where a sight distance improvement would require removal and replacement of the pavement and shoulders plus the additional earthwork. In brief, scenario 1 assumes that any pavement and shoulder costs in the project would be necessary even if the sight distance were not modified, while scenario 2 assumes that the pavement and shoulder costs are incurred only because the sight distance is being improved.

On rural two-lane highways, an improvement from AASHTO stopping sight distance criteria to those given in table 24 would cost approximately $18,000 per crest vertical curve in new construction and approximately $180,000 per crest vertical curve in rehabilitation projects. Table 26 shows that for new construction or major reconstruction projects on two-lane highways, stopping sight distance improvements for trucks are cost effective at higher average daily traffic volumes and higher truck percentages. For example, if a change in the stopping sight distance criteria for trucks could produce a 10 percent reduction in truck accidents, the improvement will be cost effective for rural two-lane highways with truck volumes over about 800 trucks/day. If a 20 percent reduction in truck accidents could be achieved, stopping sight distance improvements for trucks would be cost effective at volumes of about 400 trucks/day. On the other hand, a change in stopping sight distance criteria is almost never cost effective in rehabilitation projects on two-lane highways. Even at the extremely high volume of 15,000 veh/day and 30 percent trucks in the traffic stream (i.e., 4,500 trucks/day), a change in stopping sight distance criteria would have to reduce truck accidents by more than 20 percent to be cost effective.

On rural freeways, an improvement from AASHTO stopping sight distance criteria to those given in table 24 would cost approximately $32,000 per crest vertical curve in new construction and approximately $490,000 per crest vertical curve in rehabilitation projects. Table 27 shows that for new construction or major reconstruction projects, improved stopping sight distance criteria would be cost effective only on higher volume freeways. For example, if the stopping sight distance improvement reduced truck accidents by 10 percent, the improvement would be cost effective on rural freeways with truck volumes over about 4,000 trucks/day. If the stopping sight distance improvements reduced truck accidents by 20 percent, the improvement would be cost effective on rural freeways with truck volumes over 2,000 veh/day. By contrast, a change in stopping sight distance criteria would virtually never be cost effective in a freeway rehabilitation project. Even for a freeway with the extremely high volume of 15,000 trucks/day, a change in stopping sight distance criteria would need to reduce nearly 50 percent of truck accidents to be cost effective.

Based on these analysis results, it is recommended that the improved stopping sight distance criteria for trucks given in table 24 be considered for use in new construction or major reconstruction projects of two-lane highways that carry more than 800 trucks/day and on freeways that carry more than 4,000 trucks/day. These revised stopping sight distance criteria should be implemented with a driver eye height of 75 in (190 cm) which will provide crest vertical curves long enough to meet the needs of both passenger cars and trucks. Implementation of the stopping sight distance criteria in table 24 should not be considered in rehabilitation projects in which the sight distance improvement would require replacement of the pavement and shoulder where this would otherwise be unnecessary.

5. Summary

The sensitivity analyses performed in this study have shown that the stopping sight distance requirements of trucks are highly dependent on the assumptions concerning driver and brake system characteristics. The worst performance driver in a truck with a conventional brake system requires up to 425 ft (130 m) additional stopping sight distance than the current AASHTO criteria for a 70-mi/h (113-km/h) design speed. However, the best performance driver in a truck with a conventional brake system requires only slightly more stopping sight distance than AASHTO. Future trucks with antilock brake systems may actually require less stopping sight distance than the current AASHTO criteria.

Table 24 presents recommended stopping sight distance criteria to accommodate trucks that are cost effective only for roads with particularly high truck volumes (over 800 trucks/day on two-lane highways and over 4,000 trucks/day on freeways). The revised stopping sight distance criteria in table 24 should be used only in new construction or in major reconstruction projects where the pavement and shoulder are being replaced for reasons other than the stopping sight distance improvement; the revised criteria are not applicable to rehabilitation projects. The revised criteria in table 24 are applicable only to trucks with conventional brake systems. The existing AASHTO stopping sight distance criteria will be adequate for trucks if antilock brake systems for trucks are required by government regulations or come into nearly universal use.

B. Passing and No-Passing Zones on Two-Lane Highways

1. Current Highway Design and Operational Criteria

Two major aspects of design and operational criteria for passing and no-passing zones on two-lane highways are addressed in this section: passing sight distance and passing zone length.

a. Passing Sight Distance:

Passing sight distance is needed where passing is permitted on two-lane, two-way highways to assure that passing vehicles using the lane normally used by opposing traffic have a clear view ahead for a distance sufficient to minimize the possibility of collision with an opposing vehicle.

Design criteria: The current design criteria for passing sight distance on two-lane highways set forth in the AASHTO Green Book are based on the results of field studies conducted between 1938 and 1941 and validated by another study conducted in 1958.[56,57,58] Based on these studies, the AASHTO policy defines the minimum passing sight distance as the sum of the following four distances:

d_1 = distance traveled during perception and reaction time and during initial acceleration to the point of encroachment on the left lane.

d_2 = distance traveled while the passing vehicle occupies the left lane,

d_3 = distance between passing vehicle and opposing vehicle at the end of the passing maneuver (i.e., clearance distance), and

d_4 = distance traveled by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane, or 2/3 of d_2.

Design values for the four distances described above were developed using the field data and the following assumptions stated in the AASHTO Green Book:

- The passed vehicle travels at uniform speed.
- The passing vehicle reduces speed and trails the passed vehicle as it enters the passing section. (This is called a delayed pass.)
- When the passing section is reached, the passing driver requires a short period of time to perceive the clear passing section and to begin to accelerate.
- Passing is accomplished under what may be termed a delayed start and a hurried return in the face of opposing traffic. The passing vehicle accelerates during the maneuver, and its average speed during the occupancy of the left lane is 10 mi/h (16 km/h) higher than that of the passed vehicle.
- When the passing vehicle returns to its lane, there is a suitable clearance length between it and any oncoming vehicle in the other lane.

The design values for the four components of passing sight distance are shown in figure III-2 of the AASHTO Green Book. Table 28 illustrates the development of the design values for passing sight distance. The columns in table 28 not headed by a value of design speed represent field data from the sources cited above. The columns headed by design speeds of 20 through 70 mi/h (32 through 113 km/h) contain values that are interpolated or extrapolated from the field data presented in the intervening columns. Table 28 represents the derivation of the AASHTO passing sight distance criteria, although this derivation only appears graphically in the AASHTO Green Book.

Table 28. AASHTO passing sight distance requirements including field data used in their derivation.[1]

	Design speed (mi/h)										
	20		30		40	50		60	65		70
Assumed speed of passed vehicle (mi/h)	20		26		34	41		47	50		54
Assumed speed of passing vehicle (mi/h)	30	34.9	36	43.8	44.0	51	52.6	57	60	62.0	64
Initial maneuver:											
a = avg. acceleration (mi/h/s)	1.38	1.40	1.40	1.43	1.43	1.46	1.47	1.48	1.49	1.50	1.51
t_1 = maneuver time (s)	3.38	3.6	3.6	4.0	4.0	4.2	4.3	4.4	4.5	4.5	4.54
d_1 = distance traveled (ft)	111	145	153	216	217	275	289	325	349	366	383
Occupation of left lane:											
t_2 = time (s)	8.9	9.3	9.4	10.0	10.0	10.6	10.7	11.0	11.2	11.3	11.4
d_2 = distance traveled (ft)	393	477	497	644	648	793	827	920	985	1,030	1,075
Clearance length:											
d_3 = distance traveled (ft)	56	100	110	180	182	237	250	273	289	300	311
Opposing vehicle:											
d_4 = distance traveled (ft)	262	318	331	429	432	528	552	613	657	687	717
Total distance:											
$d_1 + d_2 + d_3 + d_4$ (ft)	800	1,040	1,100	1,470	1,500	1,800	1,920	2,100	2,300	2,380	2,500

Note: 1 mi = 1.61 km
1 ft = 0.305

It should be noted in table 28 that the speeds used to compute the design values for passing sight distance differ from the design speed of the highway. The speed of the passed vehicle is assumed to be equal to the average running speed of traffic (as represented by the intermediate volume curve in figure II-19 of the AASHTO Green Book). Thus, the speed of the passed vehicle is up to 16 mi/h (25 km/h) less than the design speed of the highway. The speed of the passing vehicle is assumed to be 10 mi/h (16 km/h) higher than the speed of the passing vehicle.

The distance traveled during the initial maneuver period (d_1) is computed by AASHTO as:

$$d_1 = 1.47\ t_1 \quad V - m + \frac{at_1}{2} \tag{31}$$

where: t_1 = time required for initial maneuver (s)

a = average acceleration (mi/h/s)

V = average speed of passing vehicle (mi/h)

m = difference in speed between passed vehicle and passing vehicle (mi/h)

The AASHTO policy estimates the time for the initial maneuver (t_1) as within the 3.6 to 4.5 s range, based on field data. Similarly, the average acceleration rate during the initial maneuver ranges from 1.38 to 1.51 mi/h/s (2.22 to 2.43 km/h/s).

The distance traveled by the passing vehicle while occupying the left lane (d_2) is estimated by AASHTO from the formula:

$$d_2 = 1.47\ Vt_2 \tag{32}$$

where: t_2 = time passing vehicle occupies the left lane (s)

V = average speed of passing vehicle (mi/h)

Based on field data, AASHTO assumes that the time the passing vehicle occupies the left lane ranges from 8.9 to 11.4 s for design speeds from 20 to 70 mi/h (32 to 113 km/h).

The clearance distance (d_3) is estimated by AASHTO to range from 33 to 310 ft (10 to 95 m), depending upon speed.

The distance traveled by an opposing vehicle (d_4) is estimated as two-thirds of the distance traveled by the passing vehicle in the left lane. Conservatively, the distances d_2 and d_4 should be equal, but the AASHTO policy assumes that the passing vehicle could abort its pass and return to the right lane if an opposing vehicle should appear early in the passing maneuver.

The bottom line in table 28 presents the AASHTO passing sight distance criteria, representing the sum of the distances d_1 through d_4. These criteria range from 800 ft (244 m) for a 20-mi/h (32 km/h) design speed to 2,500 ft (762 m) for a 70-mi/h (113 km/h) design speed. The AASHTO criteria are used in highway design to determine if a particular highway project has sufficient length with passing sight distance to assure an adequate level of service on the completed highway. The acceptable level of service for a particular project is considered to be a design decision and is not specified by the AASHTO policy. The AASHTO criteria for passing sight distance are not used in the marking of passing and no-passing zones.

Marking criteria: The criteria for marking passing and no-passing zones on two-lane highways are set by the MUTCD. Passing zones are not marked directly. Rather, the warrants for no-passing zones are established by the MUTCD, and passing zones merely happen where no-passing are not warranted. Table 29 presents the MUTCD passing sight distance warrants for no-passing zones. These criteria are based on prevailing off-peak 85th-percentile speeds rather than design speeds.

Table 29. MUTCD minimum passing sight distance warrants for no-passing zones.[2]

85th percentile speed (mi/h)	Minimum passing sight distance (ft)
30	500
40	600
50	800
60	1,000
70	1,200

Note: 1 mi = 1.61 km
1 ft = 0.305 m

The MUTCD passing sight distance warrants are substantially less than the AASHTO passing sight distance design criteria. For example, at a speed of 60 mi/h (97 km/h), the AASHTO and MUTCD passing sight distance criteria are 2,100 ft (640 m) and 1,000 ft (300 m), respectively.

The rationale for the MUTCD passing sight distance criteria is not stated in the MUTCD. However, the MUTCD warrants are identical to those presented in the 1940 AASHO policy on marking no-passing zones.[59] These earlier AASHO warrants represent a subjective compromise between distances computed for flying passes and distances computed for delayed passes. As such, they do not represent any particular passing situation. Table 30 presents the basic assumptions and data used to derive the MUTCD passing sight distance warrants.

Table 30. Derivation of MUTCD passing sight distance warrants (based on 1940 AASHTO policy).[59]

	Speed of Passing Vehicle (mi/h)				
	30	40	50	60	70
Assumed speed differential between passing and passed vehicles (mi/h)	10	12	15	20	25
Assumed speed of opposing vehicle (mi/h)	25	32	40	46	55
Required sight distance for flying pass (ft)	440	550	660	660	660
Required sight distance for delayed pass (ft)	510	760	1,090	1,380	1,780
Recommended minimum sight distance (ft)	500	600	800	1,000	1,200

Note: 1 mi = 1.61 km
1 ft = 0.305 m

b. Minimum Passing Zone Length

Another consideration in the establishment of passing and no-passing zones on two-lane highways is the minimum length of a passing zone. The AASHTO Green Book does not address passing zone lengths at all. The MUTCD indirectly sets a minimum passing zone length of 400 ft (122 m) by stating that, when two no-passing zones come within 400 ft (122 m) of one another the no-passing barrier stripe should be continued between them.

2. Critique of Highway Design and Operational Criteria

a. Passing Sight Distance

There is a clear incompatibility between the AASHTO and MUTCD passing sight distance criteria. The design values for the individual component distances in the AASHTO criteria are questionable because, at high speeds, they are based on vehicle speeds less than the design speed of the highway. On the other hand, the definition of passing sight distance as the sum of the four distance elements (d_1 through d_4) is extremely conservative, since it assumes that very early in the passing maneuver, the passing driver is committed to complete the pass. In fact, observation of two-lane highway operations shows that passing drivers frequently abort passing maneuvers.

The MUTCD passing sight distance criteria are based on a questionable premise, since they represent a compromise between delayed passes and flying passes. Furthermore, both the AASHTO and MUTCD criteria are based on field data collected nearly 50 years ago. These field studies considered only passenger cars and do not consider passing maneuvers involving longer and less powerful vehicles such as trucks. Neither the AASHTO or MUTCD models for passing sight distance contain a vehicle length term that could be used to examine the sensitivity of passing sight distance requirements to the differences between trucks and passenger cars.

Over the last 2 decades, researchers have recognized the inconsistencies between the AASHTO and MUTCD policies and have investigated alternative formulations of passing sight distance criteria. In 1971, two studies independently recognized that a key stage of a passing maneuver occurs at the point where the passing driver can no longer safely abort the pass and is, therefore, committed to complete it. One study called this the "point of no return" and another called it the "critical position."[60,61,62] A 1976 paper added the insight that the critical position is the point at which the sight distances required to abort the pass and to complete the pass are equal.[63] Until the critical position is reached, the passing vehicle can abort the pass and return to the right lane behind the passed vehicle. Beyond the critical position, the driver is committed to complete the pass, because the sight distance required to abort the pass is greater than the sight distance required to complete the pass. The critical position concept has also been incorporated in research on passing sight distance requirements published in 1982 and 1983.[64,65]

Each of the studies cited above (references 60, 61, 62, 63, 64, and 65) formulated a passing sight distance model based on the critical position concept. However, each of these models contained one or more logical flaws that made the model invalid. In 1988, however, Glennon formulated a new passing sight distance model that accounts for the kinematic relationships between the passing, passed, and opposing vehicles.[66] The location of the critical position is determined as:

$$\Delta_c = L_p + 1.47\, m \left[\frac{(2.93\, m + L_I + L_p)}{1.47\,(2V - m)} - \frac{\sqrt{4\, V\, (2.93\, m + L_I + L_p)}}{d\,(2V - m)}\right] \qquad (33)$$

where: Δ_c = critical separation (distance from front of passing vehicle to front of passed vehicle at critical position) (ft)

V = speed of passing vehicle and opposing vehicle (mi/h)

m = speed difference between passed vehicle and passing vehicle (mi/h)

d = deceleration rate used in aborting a passing maneuver (ft/s^2)

L_P = length of passing vehicle (ft)

L_I = length of passed vehicle (ft)

When the location of the critical position is known, the critical passing sight distance can be computed as:

$$PSD_C = 2\ V \left[2.93 + \frac{L_P - \Delta_C}{1.47\ m} \right] \quad (34)$$

The assumptions of the Glennon model are:

- The maximum sight distance during a passing maneuver is required at the critical position at which the sight distances required to complete the pass or to abort the pass are equal.
- The speeds of the passing vehicle and opposing vehicle are equal.
- The passing vehicle has sufficient acceleration capability to attain the specified speed difference relative to the passed vehicle by the time it reaches the critical position.
- If the passing vehicle completes its pass, it returns to its normal lane with a 1-s gap in front of the passed vehicle.
- If the passing vehicle aborts its pass, it returns to its normal lane with a 1-s gap behind the passed vehicle.
- The minimum clearance time between the passing vehicle and an opposing vehicle is 1 s.

The derivation of the Glennon model, as given in equations (33) and (34), is presented in the literature and will not be repeated here.[66]

The Glennon model combined with accepted enforcement practices provides a very safety-conservative approach for marking passing and no-passing zones on two-lane highways. If the passing sight distance determined from equation (34) is available throughout a passing zone, then it is assured that a passing driver in the critical position at any point within that zone (even at the very end) has sufficient sight distance to complete the passing maneuver safely . In most terrain, passing sight distance substantially greater than the minimum will be available throughout most of the passing zone. It must always be recognized that some drivers will illegally start a passing maneuver before the beginning of a passing zone (jumping) or complete it beyond the end of the zone (clipping). However, since the sight distance requirements of passing drivers are lower in the early and later stages of a passing maneuver than at the critical position, the model provides assurance that jumping and clipping drivers are unlikely to be greatly at risk of collision with an opposing vehicle. Finally, it should be recognized that the assumptions for a critical passing situation given above (e.g., passing and opposing vehicle traveling at the design speed of the highway, 1-s clearance time to an opposing vehicle, etc.) represent an extremely rare combination of events that does not occur often on two-lane highways.

An advantage of the Glennon model is that the length of the passing and passed vehicles appear explicitly so that the sensitivity of the required passing sight distance to vehicle length can be examined.

b. Minimum Passing Zone Length

The MUTCD minimum passing zone length of 400 ft (122 m) is clearly inadequate for high-speed passes. A 1970 study evaluated several very short passing zones.[67] In two passing zones with lengths of 400 and 640 ft (122 and 195 m), it was found that very few passing opportunities were accepted in such short zones and, of those that were accepted, more than 70 percent resulted in a slightly forced or very forced return to the right lane in the face of opposing traffic.

The 1971 study recommended that the minimum length of a passing zone should be the sum of the perception-reaction distance (d_1) and the distance traveled while occupying the left lane (d_2).[62] Table 31 illustrates several alternative criteria that could be used for the minimum length of a passing zone, including: the implicit MUTCD criteria, the sum of distances d_1 and d_2 based on the assumptions in AASHTO policy, and the 85th percentile value of the sum of distances d_1 and d_2 based on field observations.[62]

Table 31. Alternative criteria for minimum length of passing zones on two-lane highways.

Design speed (mi/h)	Minimum length of passing zone (ft): Based on MUTCD criteria	Based on $d_1 + d_2$ from AASHTO policy	Based on 85th percentile $d_1 + d_2$ observed in field studies[62]
20	400	505	-
30	400	650	-
40	400	865	-
50	400	1,065	-
55	400	1,155	885
60	400	1,245	-
65	400	1,340	1,185
70	400	1,455	1,335

Note: 1 mi = 1.61 km
1 ft = 0.305 m

3. Sensitivity Analyses

The design criteria for minimum passing sight distance and minimum passing zone length are sensitive to three major vehicle characteristics: vehicle length, acceleration/deceleration capabilities, and driver eye height.

a. Passing Sight Distance

The existing design and operational criteria for minimum passing sight distance are based on consideration of passenger cars as both the passing and passed vehicles. The sensitivity analysis presented below considers three other passing scenarios: a passenger car passing a truck, a truck passing a passenger car, and a truck passing another truck.

Passenger car passing truck: Neither the AASHTO nor the MUTCD models can be used to examine the sensitivity of passing sight distance requirements to vehicle length. However, a major advantage of the Glennon model is that the lengths of the passing and passed vehicles appear explicitly in the model. Therefore, this model has been used to compare the passing sight distance requirements for passenger cars and trucks.

The lengths of the vehicles in the sensitivity analyses that follow are based on the length of the AASHTO passenger car design vehicle (19 ft or 6 m) and the length of a relatively long truck (75 ft or 23 m).

In computing passing sight distance requirements with the Glennon model, presented above in equations (33) and (34), the deceleration rate (d) used by a passenger car in aborting a pass is assumed to be 8 ft/s^2 (2.4 m/s^2). This is a relatively conservative deceleration rate for a passenger car on a dry pavement, but it approaches a maximum deceleration rate in braking on a poor, wet road.

The sensitivity analysis considered two alternative sets of assumptions concerning the speeds of the passing and passed vehicles. The first set are the standard AASHTO assumptions that the passed vehicle travels at the average running speed of the highway (see table 28) and that the speed differential (m) between the passing and passed vehicles is a constant 10 mi/h (16 km/h) at all design speeds. The second set of assumptions were those proposed by Glennon, based on field data.[62,66] Glennon proposed that the passing vehicle should be assumed to travel at the design speed of the highway, but that the speed differential (m) between the passing and passed vehicles should be a function of design speed as shown in table 32.

Table 33 presents the passing sight distance requirements for a passenger car passing a truck using the Glennon model and Glennon's assumptions concerning vehicle speeds, presented above. (An alternative analysis with the standard AASHTO assumptions concerning vehicle speeds yielded very similar results.) For comparative purposes, the passing sight distance requirements for a passenger car passing another passenger car are presented in three different ways: (1) based on AASHTO policy; (2) based on the MUTCD warrants; and (3) based on the Glennon model.

Table 32. Speed differentials between passing and passed vehicles for particular design speeds.[66]

Design speed (mi/h)	Speed differential (mi/h)
30	12
40	11
50	10
60	9
70	8

Note: 1 mi = 1.61 km

Table 33. Sight distance requirements for passing by passenger cars based on Glennon model.[66]

Design or prevailing speed (mi/h)	AASHTO policy	MUTCD criteria	Required passing sight distance (ft) Passenger car passing passenger car	Passenger car passing truck
20	800	-	325	350
30	1,100	500	525	575
40	1,500	600	700	800
50	1,800	800	875	1,025
60	2,100	1,000	1,025	1,250
70	2,500	1,200	1,200	1,450

Note: 1 mi = 1.61 km
1 ft = 0.305 m

Table 33 shows that the passing sight distance requirements for passenger cars obtained from the Glennon model are very similar to the MUTCD criteria. The passing sight distance requirements for a passenger car passing a truck are 25 to 250 ft (8 to 76 m) higher than for a passenger car passing a passenger car, depending upon speed. The AASHTO Green Book sight distance requirements are much longer than any of the other criteria, because of their very conservative assumptions.

Truck passing passenger car: The passing sight distance requirements for a truck passing a passenger car can be addressed through a slight modification of the Glennon model. It is unlikely that a truck would be able to sustain a speed difference as large as a passenger car in performing a passing maneuver. No data are available on the speed differences actually used by trucks in passing, but for purposes of this analysis it will be assumed that trucks can maintain only half of the speed difference used by passenger cars. This assumption has been implemented in the following analysis by keeping the speed of the passed and opposing vehicles constant and decreasing the speed of the passing vehicle. Since the speeds of the passing and opposing vehicles are no longer equal, a revised version of the Glennon model was derived and utilized for this analysis. This revised model for passing maneuvers by trucks is equivalent to equations (33) and (34) with 0.5 ($V_p + V_o$) substituted for the V term, where:

V_p = speed of the passing vehicle (mi/h)

V_o = speed of the opposing vehicle (mi/h)

A truck is also not likely to use a deceleration rate of 8 ft/s² (0.25 g or 2.4 m/s²) in aborting a pass. This exceeds the capabilities of a typical truck with a poor performance driver on a poor, wet pavement. Therefore, a deceleration rate of 5 ft/s² (0.15 g or 1.5 m/s²), which would be a comfortable deceleration rate on a dry pavement and a critical deceleration rate for a poor performance driver on a poor, wet pavement, has been assumed.

Table 34 presents the passing sight distance requirements for a 75-ft (23-m) truck passing a 19-ft (6-m) passenger car under the assumptions discussed above. The passing sight distance requirements for a truck passing a passenger car are 25 to 425 ft (8 to 130 m) more than for a passenger car passing a passenger car, depending upon speed.

Table 34. Sight distance requirements for passing by trucks based on revised Glennon model.

Design or prevailing speed (mi/h)	AASHTO policy	MUTCD criteria	Required passing sight distance (ft)	
			Truck passing passenger car	Truck passing truck
20	800	-	350	350
30	1,100	500	600	675
40	1,500	600	875	975
50	1,800	800	1,125	1,275
60	2,100	1,000	1,375	1,575
70	2,500	1,200	1,625	1,875

Note: 1 mi = 1.61 km
1 ft = 0.305 m

Truck passing truck: The passing sight distance requirements for a truck passing a truck have also been examined and are also presented in table 34. Both vehicles are assumed to be 75 ft (23 m) in length. The passing sight distance requirements for a truck passing another truck were found to be 25 to 675 ft (8 to 206 m) longer than for a passenger car passing a passenger car, depending upon speed.

Comparison of results: Figure 22 compares the passing sight distance requirements determined in the sensitivity analysis with the current AASHTO and MUTCD policies. The figure indicates that the MUTCD criteria are in good agreement with the requirements for a passenger car passing another passenger car. The other passing scenarios--passenger car passing truck, truck passing passenger car, and truck passing truck--each require progressively more sight distance, but all are substantially less than the current AASHTO criteria. Figure 23 compares the minimum passing zone lengths for the same scenarios. The development and interpretation of these curves is addressed in the discussion of minimum passing zone length which follows.

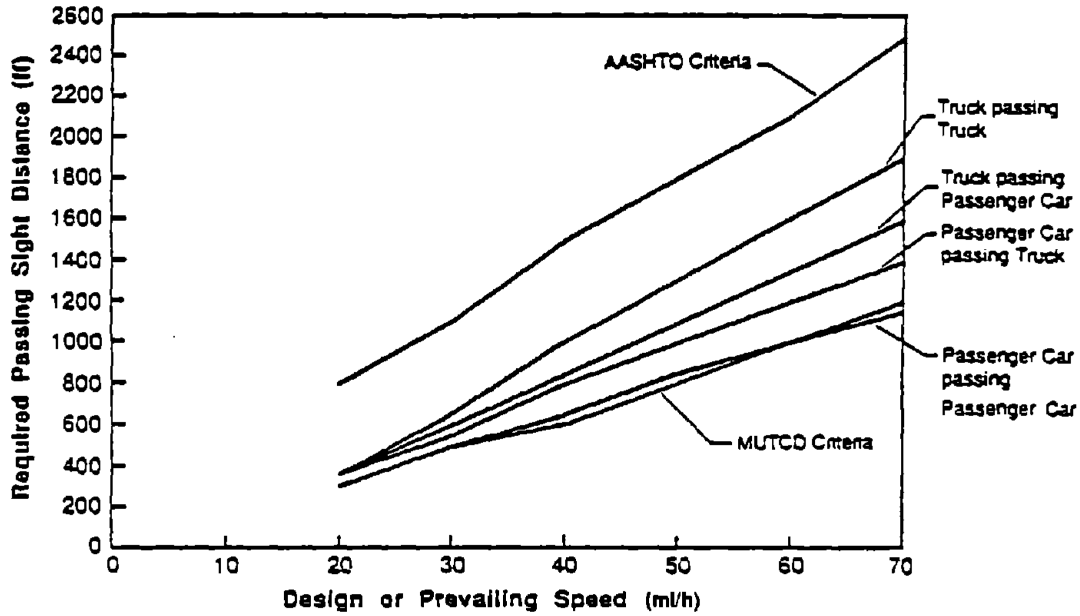
Effect of driver eye height at crest vertical curves: Where passing sight distance is restricted by a vertical curve, the truck driver has an advantage over a passenger car driver due to greater eye height. As in the case of stopping sight distance, however, the truck driver has no comparable advantage due to increased eye height where passing sight distance is restricted by a horizontal sight obstruction.

Table 35 presents the required minimum vertical curve lengths to maintain passing sight distance over a crest for the four passing scenarios addressed in tables 33 and 34. Table 35 is based on an eye height of 42 in (107 cm) for a passenger car driver and 75 in (190 cm) for a truck driver. As discussed in the sensitivity analysis for stopping sight distance in section III-A of this report, 75 in (190 cm) represents the low end of the range for truck driver eye height.

Table 35 indicates that increased driver eye height partially, but not completely offsets the greater sight distance requirements of trucks. At all speeds above 30 mi/h (48 km/h), a longer minimum vertical curve length is required to maintain adequate passing sight distance for passing maneuvers involving trucks than for a passenger car passing another passenger car. However, table 35 shows that a truck can safely pass a passenger car on any vertical curve where a passenger car can safely pass a truck.

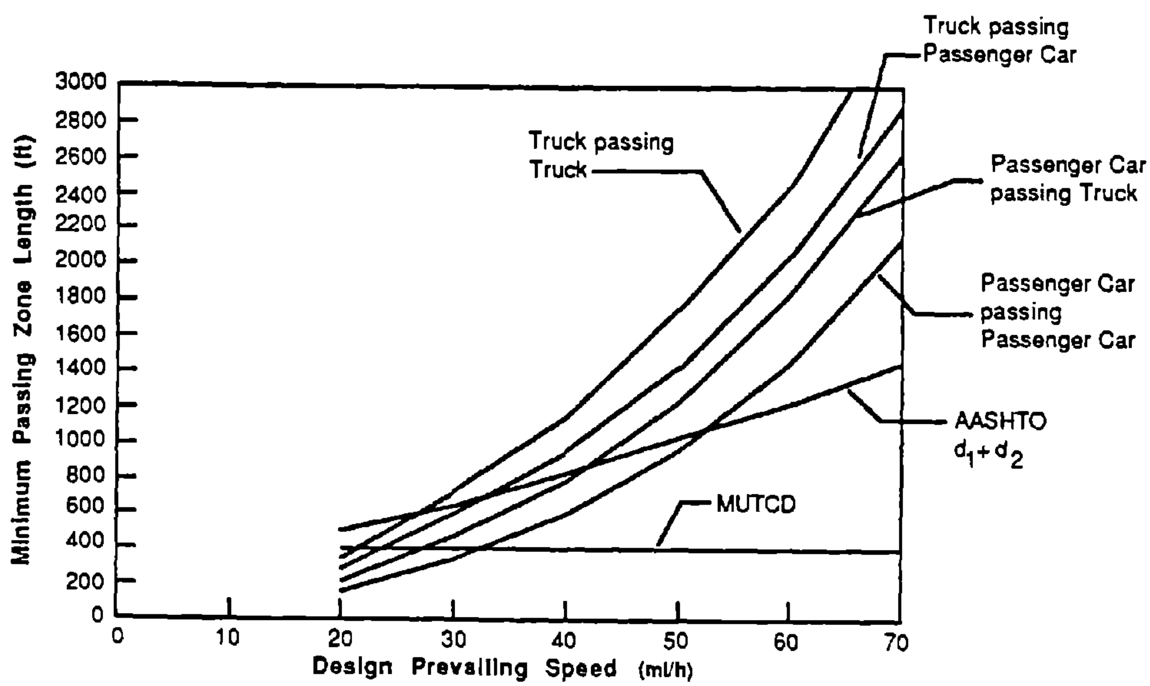
b. Minimum Passing Zone Length

There are currently no design or operational criteria for minimum passing zone length, other than the default 400-ft (122-m) guideline set by the MUTCD. One possible criterion for minimum passing zone length is the distance required for a vehicle traveling at or near the design speed of the highway to pass a slower vehicle. Recent debate over the role of trucks in passing sight distance criteria has largely ignored the longer passing distances and, thus, longer passing zone lengths required for passing maneuvers involving trucks.



Note: 1 ft = 0.305 m
1 mi = 1.61 km

Figure 22. Required passing sight distance for passenger cars and trucks in comparison to current criteria.



Note: 1 ft = 0.305 m
1 mi = 1.61 m

Figure 23. Required passing zone length to complete a pass at or near the highway design speed.

Table 35. Minimum vertical curve length (ft) to maintain required passing sight distance.

Algebraic difference in grade (%)	Design speed (mi/h) 20	30	40	50	60	70
Passenger Car Passing Passenger Car[a]						
2	80	200	350	550	760	1,030
4	160	400	700	1,100	1,510	2,060
6	230	600	1,050	1,650	2,260	3,090
8	310	790	1,400	2,190	3,010	4,120
10	380	990	1,750	2,740	3,760	5,150
Passenger Car Passing Truck[a]						
2	90	240	460	760	1,120	1,510
4	180	480	920	1,510	2,240	3,010
6	270	710	1,380	2,260	3,350	4,510
8	350	950	1,830	3,010	4,470	6,010
10	440	1,190	2,290	3,760	5,590	7,510
Truck Passing Passenger Car[b]						
2	70	190	410	670	990	1,390
4	130	380	810	1,330	1,980	2,770
6	200	570	1,210	1,990	2,970	4,150
8	260	760	1,610	2,650	3,960	5,530
10	330	950	2,010	3,320	4,950	6,910
Truck Passing Truck[b]						
2	70	240	500	860	1,300	1,840
4	130	480	1,000	1,710	2,600	3,680
6	200	720	1,500	2,560	3,900	5,530
8	260	920	1,990	3,410	5,200	7,370
10	330	1,200	2,490	4,260	6,500	9,210

[a] Based on sight distance requirements from table 33 for passenger car driver eye height of 42 in (107 cm).

[b] Based on sight distance requirements from table 34 for truck driver eye height of 75 in (190 cm).

Note: 1 ft = 0.305 m
1 mi = 1.61 km
1 in = 2.54 cm

A sensitivity analysis of passing distances has been conducted based on the following assumptions:

- The distance required to complete a pass is the sum of the initial maneuver distance (d_1) and the distance traveled in the left lane (d_2).

- The passing driver does not begin to accelerate in preparation for the passing maneuver until the beginning of the passing zone is reached.

- The initial maneuver distance (d_1) for passes by both passenger cars and trucks can be determined using the AASHTO relationship presented in equation (31). The passing vehicle is assumed to accelerate at a constant rate (a) until the desired speed differential (m) relative to the passed vehicle is reached. Thus, t_1 can be calculated as m/a.

- The acceleration rate (a) and initial maneuver time (t_1) for passes by passenger cars as a function of design speed can be approximated by the AASHTO estimates in table 28. Due to the lower performance capabilities of trucks, their acceleration rates during the initial maneuver are assumed to be 1/2 of those used by passenger cars.

- The distance traveled in the left lane (d_2) can be estimated as:

$$d_2 = V\left(\frac{2.93\ (V - m) + L_P + L_I - \frac{0.73\ m^2}{a}}{m}\right) \qquad (35)$$

  This relationship is used in preference to the AASHTO expression for d_2 because it explicitly contains the lengths of the passing and passed vehicles (L_P and L_I) and the speed difference between the vehicles (m). It would be desirable to calibrate equation (35) with field data.

- Equation (35) is based on the premise that the passing vehicle initially trails the passed vehicle by a 1-s gap and returns to its normal lane leading the passed vehicle by a 1-s gap. The passing vehicle is assumed to maintain an average speed differential equal to m during its occupancy of the left lane; the latter assumption is consistent with AASHTO policy, but is more restrictive than the Glennon model, which assumes only that a speed differential equal to m is reached before the passing vehicle reaches the critical position.[66]

- Passenger cars are assumed to accelerate when passing and maintain an average speed equal to the design speed of the highway and maintain the same average speed differences used to derive table 33. When passing, trucks are assumed to maintain only half of the speed difference of passenger cars, consistent with the assumptions used to derive table 34.

- The assumed lengths of passenger cars and trucks are 19 and 75 ft (6 and 23 m), respectively.

The sensitivity analysis results for the distance required to complete a pass are presented in table 36 for the four passing scenarios considered previously--passenger car passing passenger car, passenger car passing truck, truck passing passenger car, and truck passing truck. The required passing distances for these four scenarios are illustrated in figure 23. Except at very low speeds, all of the passing distances are very much larger than the MUTCD minimum passing zone length of 400 ft (122 m).

Table 36. Passing zone length required to complete a pass for various passing scenarios.

Design speed (mi/h)	Passing vehicle speed (V) (mi/h)	Speed difference (m) used by passing vehicle		Minimum length of passing zone (ft)			
		Passenger car	Truck	Car passing passenger	Passenger car passing truck	Truck passing passenger car	Truck passing truck
20	20	13	6.5	150	225	275	350
30	30	12	6	350	475	600	725
40	40	11	5.5	600	825	975	1,175
50	50	10	5	975	1,250	1,450	1,750
60	60	9	4.5	1,475	1,850	2,025	2,450
70	70	8	4	2,175	2,650	2,900	3,400

Note: 1 mi = 1.61 km
1 ft = 0.305 m

Table 36 and figure 23 show that in order to complete a passing maneuver at speeds of 60 mi/h (97 km/h) or more under the stated assumptions, trucks require passing zones at least 2,000 ft (610 m) long. There are relatively few such passing zones on two-lane highways and, yet, trucks regularly make passing maneuvers. The explanation of this apparent paradox is that, since there are very few locations where a truck can safely make a delayed pass, truck drivers seldom attempt them. Most passing maneuvers by trucks on two-lane highways are flying passes that require less passing sight distance and less passing zone length than delayed passes. Thus, there may be no need to change current passing sight distance criteria to accommodate a truck passing a passenger car or a truck passing a truck as shown in table 34. It makes little sense to provide enough passing sight distance for delayed passes by trucks when passing zones are not generally long enough to permit such maneuvers.

4. Recommended Revisions to Design and Operational Criteria

There is very close agreement between the current MUTCD criteria for passing sight distance and the sight distance requirements for a passenger car passing another passenger car based on an analytical model recently developed by Glennon.[66] Application of the Glennon model indicates that successively longer passing sight distances are required for a passenger car passing a truck, a truck passing a passenger car, and a truck passing a truck. There is no general agreement as to which of these passing situations is the most reasonable basis for designing and operating two-lane highways. All of the passing sight distance criteria derived here are shorter than the AASHTO design criteria, which are based on very conservative assumptions.

The analysis results indicate that, if a passenger car passing a passenger car is retained as the design situation, only minor modifications are needed to the MUTCD passing sight distance criteria. If a more critical design situation is selected (e.g., a passenger car passing a truck), passing sight distances up to 250 ft (76 m) longer than the current MUTCD criteria would be required. It is important to recognize that such a change in passing zone marking criteria would completely eliminate some existing passing zones and shorten others, even though passenger cars can safely pass other passenger cars in those zones. Clearly, this would reduce the level of service on two-lane highways.

No cost-effectiveness analysis of the potential for revising passing sight distance criteria to accommodate trucks was conducted because of the lack of data on the operational effects of implementing the revised criteria. Thus, a formal recommendation as to whether the revised passing sight distance criteria for trucks in tables 33 and 34 should be adopted would be premature. The operational effects of remarking passing and no-passing zones on two-lane roads could be investigated with existing computer simulation models. A cost-effectiveness analysis could then be undertaken using the general approach presented in appendix F in volume II of this report to determine the percentage reduction in truck accidents on two-lane roads that would be required to offset the cost of removing and replacing the centerline markings plus the operational disbenefit of the revised markings for passenger cars (i.e., fewer passing maneuvers and increased delay).

5. Summary

Alternatives to the current MUTCD criteria for passing sight distance on two-lane highways have been developed. The criteria, presented in tables 33 and 34, address design situations involving a passenger car passing a truck, a truck passing a passenger car, and a truck passing a truck, in contrast to the current criteria which are based on a passenger car passing a passenger car. Adoption of any of these alternative passing sight distance criteria for marking passing and no-passing zones on two-lane highways would be premature without an operational analysis of the extent to which the revised criteria would degrade the level of service for passenger cars.

The increased driver eye height of trucks partially, but not completely, offsets the increased passing sight distance requirements when the truck is the passing vehicle. However, except at very sharp crests on high-speed highways, a truck can safely pass a passenger car on any crest where a passenger car can safely pass a truck.

There are no current criteria for passing zone lengths, except for the default 400-ft (122-m) guideline set by the MUTCD. For all design speeds above 30 mi/h (48 km/h), the distance required for one vehicle to pass another at or near that design speed is substantially longer than 400 ft (122 m), indicating a need for longer passing zones. The required passing distances and passing zone lengths are increased substantially when the passing vehicle, the passed vehicle, or both, are trucks.

C. Decision Sight Distance

1. Current Design and Operational Criteria

Decision sight distance is the distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source or hazard in a roadway environment that may be visually cluttered, to recognize the hazard or its threat potential, select an appropriate speed and path, and initiate and complete the selected maneuver safely and efficiently. Decision sight distance is intended to give drivers an additional margin for error and to provide them sufficient length to complete their selected maneuver at the same or reduced speed, rather than to stop. Therefore, the recommended values of decision sight distance are substantially longer than the recommended stopping sight distance criteria. Locations where decision sight distance may be needed include: interchanges and intersections, locations where unusual or unexpected maneuvers are required; changes in cross-section such as toll plazas and lane drops; and areas of "visual noise" where multiple sources of information, such as roadway elements, traffic, traffic control devices, and advertising signs, compete for the driver's attention. The decision sight distance criteria recommended in the AASHTO Green Book are presented in table 37. Table 37 also documents the components considered in the derivation of decision sight distance, which are discussed below.

Vertical curve lengths to provide these levels of decision sight distance are based on a 42-in (107-cm) driver eye height and a 6-in (15-cm) object height, just as for stopping sight distance. Table 38 presents the minimum vertical curve lengths required to achieve the AASHTO criteria for decision sight distance for a range of design speeds and algebraic differences in grade. The minimum vertical curve lengths were obtained by using the higher value of decision sight distance in table 37 for each design speed in equations (27) and (28).

Table 37. AASHTO criteria for decision sight distance.[1]

Design speed (mi/h)	Time (s): Premaneuver: Detection and recognition	Time (s): Premaneuver: Decision and response initiation	Time (s): Maneuver (lane change)	Time (s): Total	Decision sight distance (ft): Computed	Decision sight distance (ft): Rounded for design
30	1.5-3.0	4.2-6.5	4.5	10.2-14.0	449-616	450-625
40	1.5-3.0	4.2-6.5	4.5	10.2-14.0	598-821	600-825
50	1.5-3.0	4.2-6.5	4.5	10.2-14.0	748-1,027	750-1,025
60	2.0-3.0	4.7-7.0	4.5	11.2-14.5	986-1,276	1,000-1,275
70	2.0-3.0	4.7-7.0	4.0	10.7-14.0	1,098-1,437	1,100-1,450

Note: 1 mi = 1.61 km
1 ft = 0.305 m

Table 38. Required minimum vertical curve length (ft) to provide maximum AASHTO decision sight distance for passenger cars.
(Driver eye height = 42 in)

Algebraic difference in grade (%)	Design speed (mi/h): 30	40	50	60	70
2	590	1,030	1,590	2,450	3,170
4	1,180	2,050	3,170	4,900	6,330
6	1,770	3,080	4,750	7,340	9,500
8	2,360	4,100	6,330	9,790	12,660
10	2,940	5,130	7,910	12,240	15,820

Note: 1 ft = 0.305 m
1 in = 2.54 cm
1 mi = 1.61 km

The AASHTO decision sight distance criteria are meant to be guidelines rather than absolute requirements. The AASHTO Green Book emphasizes the importance of traffic control devices, such as advance signing, where the full decision sight distance cannot be provided. This issue is addressed further in Section III-P of this report on sign placement.

2. Critique of Design and Operational Policy

The AASHTO criteria for decision sight distance are based on a 1978 FHWA study.[68] The factors considered in the development of decision sight distance criteria in that study are discussed here.

In the AASHTO criteria, decision sight distance is based on the time required for three phases of the decision and maneuver process--two premaneuver phases and the maneuver itself.

- The first premaneuver phase is detection and recognition. These two elements of the information handling process include time periods for latency (the delay between the time a hazard is presented and the time that the driver's eyes begin to move toward it), eye movement to hazard, eye fixation, and finally, recognition or perception of the hazard. Times up to 3 s have been reported for this process.[69]

- The second premaneuver phase is decision and response initiation. Once the hazard is perceived, the driver needs to identify alternative maneuvers, select one, and then initiate the required action. The complex decisions potentially required on the highway could require a variety of driver responses. The AASHTO criteria are based on one particular response--a lane change maneuver. The time required to decide on this maneuver, search for gaps in traffic to enter the adjacent lane, and initiate the lane change maneuver is estimated to be 4.2 to 7.0 s.[70] It seems logical that the time required to search for a gap would increase with traffic volume. While the AASHTO Green Book recognizes this, it specifically excludes traffic volume effects on gap-search times from the time allowed for decision and response initiation. Thus, the current decision sight distance criteria are only applicable to low traffic volume conditions.

- The final component of decision sight distance is the lane change maneuver itself. This maneuver is expected to require 4.0 to 4.5 s, based on field studies.[68]

The estimated times for these phases of the decision and maneuver process are shown in table 37, as a function of design speed. The recommended decision sight distance criteria represent the distance traveled by a vehicle at the design speed during the time interval corresponding to the three phases of the decision and maneuver process.

The estimates of the times required for all three components of decision sight distance are based on studies that considered passenger cars alone and did not consider trucks. There are no reliable data to indicate the times required for trucks to complete each phase of the decision and maneuver process.

The AASHTO Green Book recommends the application of the decision sight distance criteria in table 37 at a wide variety of locations where increased sight distance may be needed. The types of locations that may need decision sight distance have been listed in the discussion of the current criteria. However, the numerical values for the current criteria are based on a single type of maneuver--a lane change approaching a major fork on a freeway. While the concept of decision sight distance has broad application to many portions of the highway system, no single set of numerical criteria can address all of these potential situations. In other words, the appropriate value of decision sight distance should vary with the type of location and the type of maneuver required.

3. Sensitivity Analysis

Since there are no data on the times required by trucks for the three components of decision sight distance, a sensitivity analysis of the differences between design criteria for passenger cars and trucks must necessarily be based on assumptions concerning truck and truck driver performance. The following analysis of the three components of decision sight distance examines the potential differences between passenger cars and trucks:

- The detection and recognition phase is based on the human capabilities of drivers. There are no data to presume that there is any difference between passenger car and truck drivers in detecting and recognizing hazards. (In fact, professional truck drivers may arguably have better detection and recognition times than passenger car drivers.)

- The decision and response initiation phase requires the driver to identify the need for a lane change maneuver, find a suitable gap, and initiate a lane change maneuver to enter that gap. Trucks obviously require longer gaps than passenger cars and, under high traffic volume conditions, a truck driver will clearly require more time than a passenger car driver to locate a adequate gap in the adjacent lane. However, the AASHTO Green Book criteria do not consider the effect of high traffic volume conditions on the search for gaps by passenger car drivers, so it is not appropriate to consider high volume conditions for trucks either. Therefore, the decision and response initiation times for truck drivers were assumed to be the same as for passenger car drivers. The decision sight distance criteria considered here for both passenger cars and trucks should be clearly understood to be based on low traffic volume conditions as the design situation.

- Trucks are longer and wider than passenger cars and undoubtedly require more time than passenger cars to change lanes. However, there are no data to indicate how much additional time trucks require to change lanes. For purposes of this sensitivity analysis, two alternative assumptions will be made: (1) that trucks require more time than passenger cars to change lanes in proportion to their increased width (8.5 ft [2.6 m] versus 7 ft [2.1 m] or a 21 percent increase); and, (2) that trucks require twice as long to change lanes as passenger cars. The second assumption is very conservative in comparison to the first.

Tables 39 and 40 present the decision sight distance criteria for trucks derived on the basis of the two alternative assumptions concerning the time required for trucks to change lanes. Figure 24 compares the minimum and maximum decision sight distance requirements for passenger cars and trucks, as a function of design speed. The top portion of figure 24 represents the lower values of decision sight distances given in tables 37, 39, and 40, while the bottom portion of the figure represents the higher values of decision sight distance.

Where sight distance is restricted by a vertical crest, increased driver eye height provides trucks with an advantage over passenger cars. Table 41 presents the required minimum vertical curve lengths to achieve these decision sight distances, as a function of design speed, for both the minimum (75 in or 190 cm) and average (93 in or 236 cm) values of truck driver eye height. A comparison of tables 38 and 41 indicates that for truck maneuver times 21 percent greater than for passenger cars, the vertical curve lengths required to maintain decision sight distance for trucks are always less than those required for passenger cars. This finding also applies to truck maneuver times 100 percent greater than for passenger cars with a truck driver eye height of 93 in (236 cm) but, in this case, trucks require longer vertical curves than passenger cars if the truck driver eye height is lowered to 75 in (190 cm).

At horizontal sight restrictions, the increased driver eye height of trucks provides no advantage over passenger cars, unless the sight restriction is a low object that the truck driver can see over. In most cases, trucks will have no advantage over passenger cars and the full decision sight distances shown in figure 24 will be needed.

4. Recommended Revisions to Design and Operational Criteria

Tables 39 and 40 present decision sight distance criteria to accommodate trucks as potential alternatives to the current AASHTO criteria presented in table 37. A cost-effectiveness analysis was conducted to determine whether there is likely to be economic justification for provision of increased decision sight distance for trucks. This analysis was entirely analogous to the cost-effectiveness analysis of stopping sight distance presented in section III-A and appendix F. The analysis addressed the minimum percentage reduction in accidents that would be required to provide benefits equivalent to the cost of the additional earthwork required to provide the decision sight distance specified in table 39 rather than the existing AASHTO criteria for crest vertical curves on rural freeways with design speeds of 70 mi/h (113 km/h).

Table 39. Revised criteria for decision sight distance.
(Maneuver time increased by 21 percent to allow for trucks)

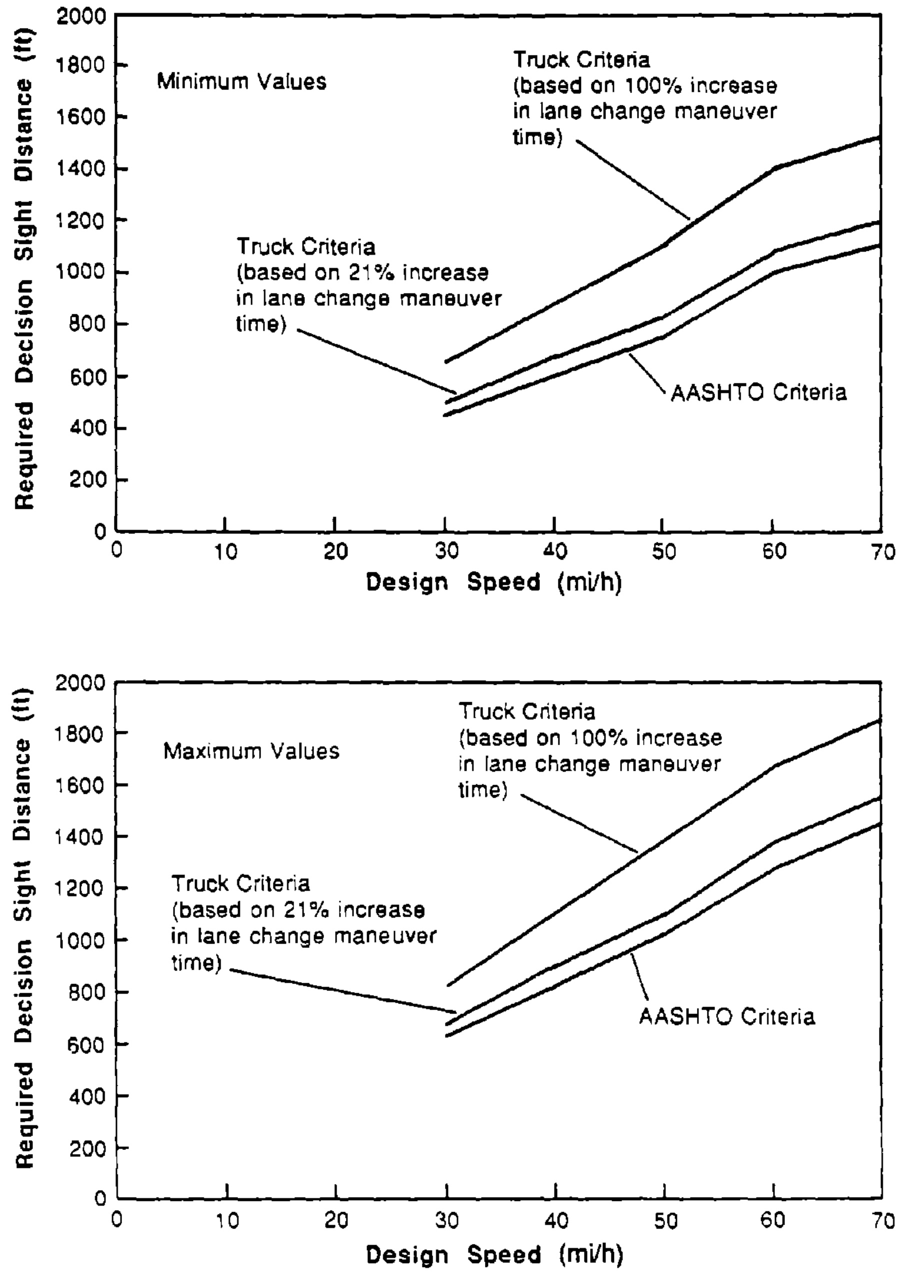
	Time (s)				Decision sight distance (ft)	
	Premaneuver					
Design speed (mi/h)	Detection and recognition	Decision and response initiation	Maneuver (lane change)	Total	Computed	Rounded for design
30	1.5-3.0	4.2-6.5	5.5	11.2-15.0	493-660	500-675
40	1.5-3.0	4.2-6.5	5.5	11.2-15.0	657-880	675-900
50	1.5-3.0	4.2-6.5	5.5	11.2-15.0	821-1,100	825-1,100
60	2.0-3.0	4.7-7.0	5.5	12.2-15.5	1,074-1,364	1,075-1,375
70	2.0-3.0	4.7-7.0	4.9	11.6-14.9	1,191-1,530	1,200-1,550

Note: 1 mi = 1.61 km
1 ft = 0.305 m

Table 40. Revised criteria for decision sight distance.
(Maneuver time increased by 100 percent to allow for trucks)

Design speed (mi/h)	Time (s): Premaneuver: Detection and recognition	Time (s): Premaneuver: Decision and response initiation	Time (s): Maneuver (lane change)	Time (s): Total	Decision sight distance (ft): Computed	Decision sight distance (ft): Rounded for design
30	1.5-3.0	4.2-6.5	9.0	14.7-18.5	647-814	650-825
40	1.5-3.0	4.2-6.5	9.0	14.7-18.5	862-1,085	875-1,100
50	1.5-3.0	4.2-6.5	9.0	14.7-18.5	1,078-1,357	1,100-1,375
60	2.0-3.0	4.7-7.0	9.0	15.7-19.0	1,382-1,672	1,400-1,675
70	2.0-3.0	4.7-7.0	8.0	14.7-18.0	1,509-1,848	1,525-1,850

Note: 1 mi = 1.61 km
1 ft = 0.305 m



Note: 1 ft = 0.305 m
1 mi = 1.61 km

Figure 24. Comparison of estimated decision sight distance for trucks with AASHTO criteria.

Table 41. Required minimum vertical curve length (ft) to provide maximum decision sight distance for trucks.

Decision sight distance criteria	Driver eye height (in)	Algebraic difference in grade (%)	Design speed (mi/h) 30	40	50	60	70
Maneuver time for trucks 21% greater than for passenger cars (see table 39)	75	2	450	790	1,180	1,840	2,340
		4	890	1,580	2,360	3,680	4,680
		6	1,330	2,370	3,530	5,520	7,010
		8	1,780	3,150	4,710	7,360	9,350
		10	2,220	3,940	5,890	9,200	11,680
Maneuver time for trucks 21% greater than for passenger cars (see table 39)	93	2	380	670	1,000	1,560	1,980
		4	750	1,330	1,990	3,110	3,950
		6	1,130	2,000	2,980	4,660	5,920
		8	1,500	2,660	3,980	6,210	7,890
		10	1,870	3,330	4,970	7,760	9,860
Maneuver time for trucks 100% greater than for passenger cars (see table 40)	75	2	670	1,180	1,840	2,730	3,330
		4	1,330	2,360	3,680	5,460	6,660
		6	1,990	3,530	5,520	8,190	9,990
		8	2,650	4,710	7,360	10,920	13,310
		10	3,310	5,890	9,200	13,640	16,640
Maneuver time for trucks 100% greater than for passenger cars (see table 40)	93	2	560	1,000	1,560	2,310	2,810
		4	1,120	1,990	3,110	4,610	5,620
		6	1,680	2,980	4,660	6,910	8,430
		8	2,240	3,980	6,210	9,210	11,240
		10	2,800	4,970	7,760	11,520	14,050

Note: 1 ft = 0.305 m
1 in = 2.54 cm
1 mi = 1.61 km

The analysis found that the required minimum percentage reduction in truck accidents for specific levels of ADT and percent trucks were approximately 4 times higher than the values presented for scenario 1 (new construction) of the stopping sight distance analysis in table 27. It is highly unlikely that decision sight distance improvements could achieve such large reductions in accidents. Therefore, modification of the current AASHTO decision sight criteria is not recommended.

5. Summary

A sensitivity analysis found that 100 to 400 ft (30 to 122 m) more decision sight distance may be required for trucks than for passenger cars at a design speed of 70 mi/h (113 km/h), depending on the assumptions made concerning the differences in lane change maneuver times between passenger cars and trucks. Smaller differences in decision sight distance requirements for passenger cars and trucks were found at lower design speeds. The increased decision sight distance needed by trucks is partially, but not completely, offset at vertical crests by their higher driver eye heights. However, driver eye height provides no comparable advantage at horizontal sight restrictions.

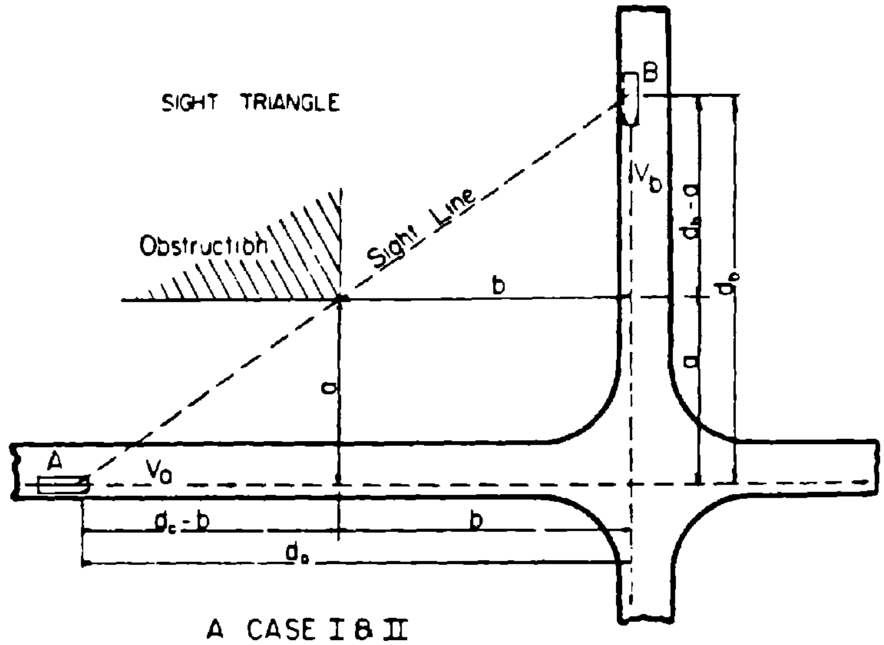
A cost-effectiveness analysis found that the provision of additional decision sight distance for trucks in new construction on rural freeways by lengthening crest vertical curves on the approaches to decision points would be cost effective only if it provided accident reduction 4 times higher than comparable stopping sight distance improvements. Such sight distance improvements were themselves found to be cost effective only at locations with very high truck volumes. Therefore, it is unlikely that it would be cost effective to change current decision sight distance criteria to accommodate trucks. Furthermore, it would not be even remotely cost effective to correct existing decision sight distance deficiencies by lengthening crest vertical curves in rehabilitation projects. Instead, it is recommended that increased emphasis should be placed on improved signing in advance of existing decision points with high truck volumes.

D. Intersection Sight Distance

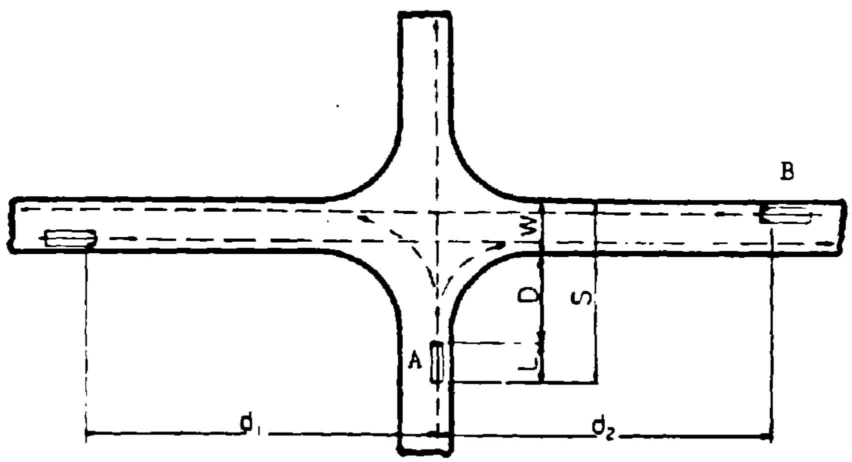
1. Current Highway Design and Operational Criteria

The 1984 AASHTO Green Book considers intersection sight distance to be adequate when an unobstructed line of sight is provided to the entire intersection and a sufficient length of the intersecting highway to permit approaching drivers to avoid collisions. The AASHTO criteria incorporate various assumptions of physical conditions and driver behavior including vehicle speed, vehicle performance capabilities, and distances traveled during perception-reaction time and locked-wheel braking.

Sight distance to be provided at intersections is determined by calculating the unobstructed sight distance for vehicles approaching simultaneously on two crossing roadways or for vehicles accelerating from a stop at an intersection approach. Figure 25 illustrates the current design considerations for these two general situations. The simultaneous approach of vehicles



A CASE I & II
NO CONTROL OR YIELD CONTROL ON MINOR ROAD



B CASE III
STOP CONTROL ON MINOR ROAD

Figure 25. Design considerations for intersection sight distance.[1]

on intersecting roadways is considered at "uncontrolled" intersections or where the minor approach has a posted YIELD sign. The consideration of acceleration from a stop assumes that a STOP sign is present on the minor roadway or traffic signalization is provided for all approaches.

AASHTO considers four general cases for establishing minimum intersection sight distance dimensions. The four conditions represent various levels of control applied to at-grade intersections:

Case I -- No control, but allowing vehicles to adjust speed.

Case II -- YIELD control where vehicles on the minor intersecting roadway must yield to vehicles on the major intersecting roadway.

Case III -- STOP control where traffic on the minor roadway must stop prior to entering the major roadway.

Case IV -- Signal control where all legs of the intersecting roadways are required to stop by either a STOP sign or where the intersection is controlled by traffic signals.

a. Case I -- No Control

The operator of a vehicle must be able to perceive a hazard in sufficient time to alter the vehicle's speed as necessary before reaching an intersection that is not controlled by YIELD signs, STOP signs, or traffic signals. The sight distance required is a function of the speed of the vehicles and the time to perceive and react by accelerating or decelerating.

The following equation represents AASHTO's method to determine the minimum sight distance along each approach:

$$ISD = 1.47\ V\ t \tag{36}$$

where: ISD = d_a or d_b; minimum intersection sight distance (ft); (see upper portion of figure 25)

V = speed of vehicle (mi/h)

t = $t_{pr} + t_r$ (s) (assumed: t = 3.0 s)

t_{pr} = perception-reaction time (s) (assumed: t_{pr} = 2.0 s)

t_r = time required to regulate speed (s) (assumed: t_r = 1.0 s)

b. Case II -- YIELD Control

The sight distance for the vehicle operator on the minor road must be sufficient to allow the operator to see a vehicle on the major roadway approaching from either the left or the right, and then bring the vehicle to a stop prior to reaching the intersecting roadway. This maneuver requires sight distance equal to the stopping sight distance specified in equation (25), which is a function of perception-reaction time and braking time.

c. Case III -- STOP Control

The AASHTO Green Book states: "Where traffic on the minor road of an intersection is controlled by STOP signs, the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position even though the approaching vehicle comes in view as the stopped vehicle begins its departure movements." This situation is illustrated in the lower portion of figure 25. There are three basic maneuvers which occur at the average intersection. These maneuvers are:

A. Traveling across the intersecting roadway by clearing traffic on both the left and the right of the crossing vehicle (case III-A in figure 26);

B. Turning left into the intersecting roadway by first clearing traffic on the left and then to enter the traffic stream with vehicles from the right (case III-B in figure 26); and

C. Turning right into the intersecting roadway by entering the traffic stream with vehicles from the left (case III-C in figure 26).

The AASHTO Green Book presents separate sight distance criteria for each case.

Case III-A -- Crossing Maneuver: As stated in the AASHTO Green Book "... the sight distance for a crossing maneuver is based on the time it takes for the stopped vehicle to clear the intersection and the distance that a vehicle will travel along the major road at its design speed in that amount of time." The sight distance may be calculated from the equation:

$$ISD = 1.47\ V\ (J + t_a) \tag{37}$$

where: ISD = d_1 or d_2 sight distance along the major highway from the intersection (ft)

V = design speed of the major highway (mi/h)

J = sum of the perception time and the time required to actuate the clutch or actuate an automatic shift (assumed: J = 2.0 s)

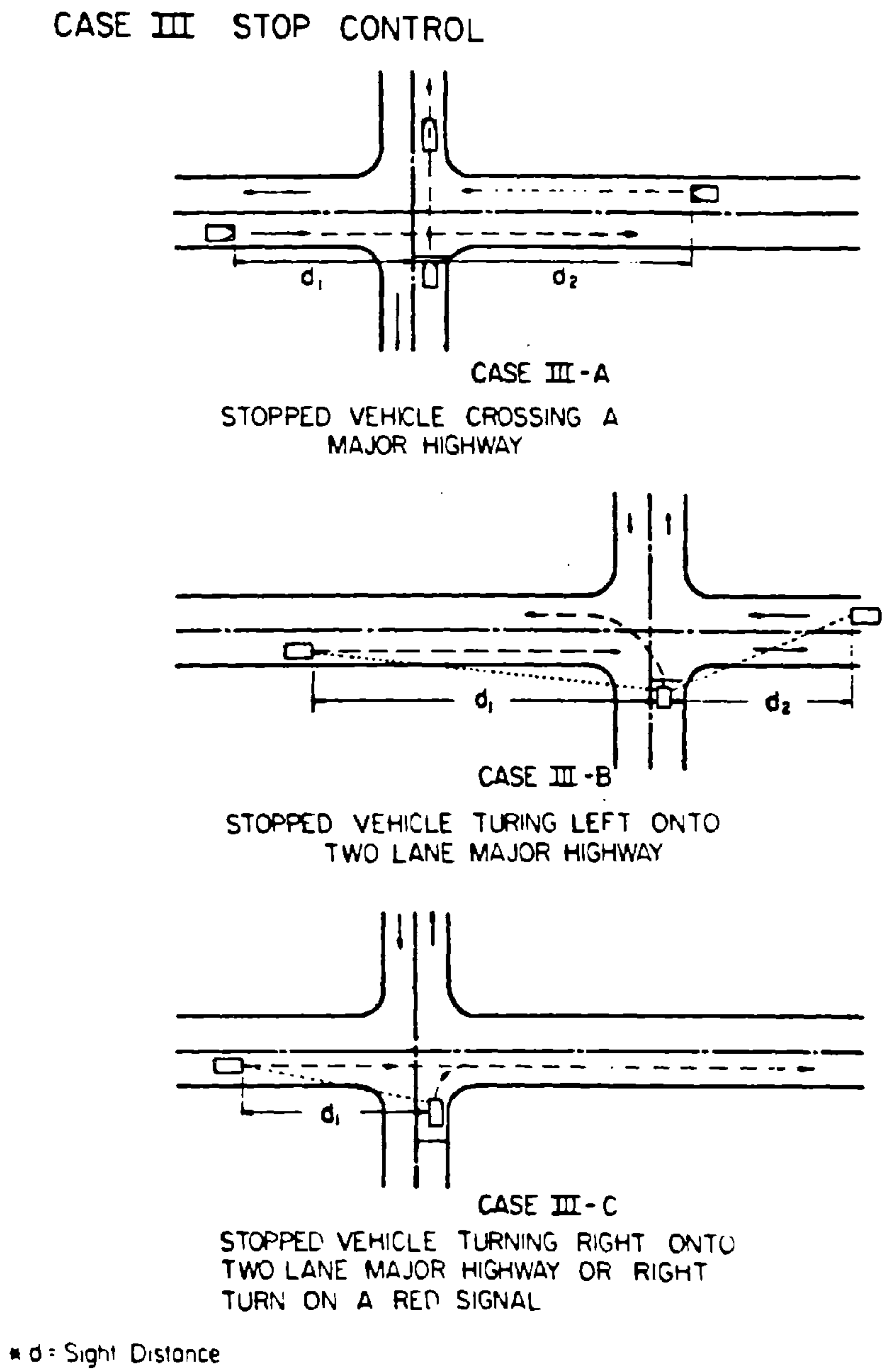


Figure 26. Intersection sight distance cases for STOP-controlled intersections.[1]

t_a = time required to accelerate and traverse the distance (S) to clear the major highway pavement (s) (Note: Values of t_a can be read directly from figure IX-21 in the AASHTO Green Book for nearly level conditions for a given distance S)

S = D + W + L, the distance that the crossing vehicle must travel to clear the major highway (ft). (See the lower portion of figure 25.)

D = distance from the near edge of pavement to the front of a stopped vehicle (assumed: D = 10 ft or 3 m)

W = pavement width along path of crossing vehicle (ft)

L = overall length of vehicle (ft) (Note: AASHTO Green Book values are 19, 30, 50, 55, and 65 ft [6, 9, 15, 17, and 20 m] for the P, SU, WB-40, WB-50, and WB-60 vehicles, respectively)

<u>Case III-B -- Turning Left into a Crossroad</u>: A vehicle turning left into a cross road should have, as a minimum, sight distance to a vehicle approaching from the right traveling at the design speed. The turning vehicle should be able to accelerate to the average running speed by the time the approaching vehicle gets within a specified tailgate distance or minimum separation after reducing its speed to the average running speed, or the turning vehicle should be able to accelerate up to the design speed by the time the approaching vehicle gets within the specified tailgate distance maintaining the design speed. Figure IX-24 in the AASHTO Green Book illustrates the details of this case.

AASHTO states that the required sight distances for trucks making left turns onto a crossroad will be substantially longer than for passenger cars. AASHTO further indicates that the sight distance for trucks can be determined using appropriate assumptions for vehicle acceleration rates and turning paths. The specific assumptions, however, are not documented in the Green Book. Thus, this case, as presented by AASHTO, lacks sufficient information to derive the design curves for determining the required sight distances.

<u>Case III-C -- Turning Right into a Cross Road</u>: A right turning vehicle must have sufficient sight distance to vehicles approaching from the left to complete its right turn and to accelerate to the running speed of the major roadway before being overtaken by traffic approaching the intersection from the left and traveling at the same running speed. The case III-C criteria are documented in figure IX-25 of the AASHTO Green Book. As in case III-B, AASHTO indicates that the sight distances for trucks need to be considerably longer than for passenger cars, but sufficient information is lacking to derive the design curves presented in the Green Book.

d. Case IV -- Signal Control

Due to the increased workload present at an intersection, the AASHTO Green Book recommends that drivers accelerating at a signalized intersection should have sight distances available based on the case III procedures. Hazards associated with vehicles turning or crossing an intersection strengthen the argument for providing the case III sight distance. The AASHTO rationale for providing sight distance at signalized intersections equivalent to case III is that motorists should have sufficient sight distance to (1) be able to see the traffic signal in sufficient time to perform the action it indicates; (2) have a view of the intersecting approaches in case a crossing vehicle violates the signal indication or in case the signal malfunctions; and (3) have a sufficient departure sight line for a right-turn-on-red maneuver.

e. Effect of Grades

The AASHTO case II intersection sight distance criteria indicate that approach grades up to 3 percent have little effect on stopping sight distances, and grades up to 6 percent may be ignored if great precision is not desired. However, case III is materially affected by the grade of approach on the minor road. Trucks are more sensitive to approach grades than passenger cars because, while a passenger car may start on a level approach, a truck could have its rear axle(s) on the grade. Table 42 lists the multiples to be applied to t_a (i.e., time required to cross the major highway) to adjust for grades on the minor highway. The AASHTO Green Book does not provide a supporting framework for these values.

Table 42. AASHTO adjustment factor for the effect of crossroad grade on accelerating time at intersections.[1]

Design vehicle	Crossroad grade (percent) -4	-2	0	+2	+4
P	0.7[a]	0.9	1.0	1.1	1.3
SU	0.8	0.9	1.0	1.1	1.3
WB-50	0.8	0.9	1.0	1.2	1.7

[a] Note: Each adjustment factor is the ratio of acceleration time on the grade to acceleration time on the level.

f. Sight Distance at Ramp Terminals

Each principle discussed above is applicable to the design of at-grade intersections which are ramp terminals. An added sight distance consideration at a ramp terminal is the location of bridge parapet walls and/or bridge railings. Sight distance criteria for ramp terminals are intended to assure that a vehicle stopped at the ramp terminal will have adequate time to turn left and clear the intersection without colliding with a vehicle coming from the left. Table 43 lists the sight distance requirements at various design speeds for three classes of vehicles (P, SU, WB-50).

Table 43. AASHTO criteria for sight distance along the crossroad for an at-grade ramp terminal.[1]

Assumed Design Speed on Crossroad Through the Interchange	Sight Distance Required to Permit Design Vehicle to Turn Left from Ramp to Crossroad (ft)[a] — Design Vehicle Assumed at Ramp Terminal			Sight Distance Available to Entering Vehicle When Vertical Curve on Crossroad Is Designed for Stopping Sight Distance[b]	
	P	SU	WB-50	P	SU or WB-50
70	740	1,060	1,430	920	1,040
60	630	910	1,230	730	820
50	530	760	1,030	540	600
40	420	610	820	420	480
30	320	460	620	310	350

[a] Sight distance measured from height of eye of 3.50 ft for P, SU, and WB-50 design vehicles to an object 4.25 ft high.

[b] Minimum available stopping sight distance based on the assumption that there is no horizontal sight obstruction and that $S < L$.

Note: 1 ft = 0.305 m

The primary difference between these criteria and the typical case III situation is the increase in the time and distance traveled by vehicles negotiating the left turn rather than crossing the highway. The distances assumed by AASHTO for the vehicle to clear the intersection are 60 ft (18 m) for the P design vehicle, 90 ft (27 m) for the SU design vehicle, and 120 ft (37 m) for WB-50 design vehicle. Other assumptions include:

- The front of the stopped vehicle is 10 ft (3 m) from the edge of the through pavement (i.e., D = 10 ft or 3 m).
- The turning vehicle follows its minimum turning path.
- The turning vehicles enters a two-lane, two-way highway.
- The time to accelerate is the same as for case III-A (see figure IX-21 in the AASHTO Green Book.
- The perception and preparation time is 2.5 s.

The AASHTO criteria indicate that both the horizontal sight triangle and the vertical curvature should be checked to ensure that the required "critical" sight distance from table 43 is provided. Further, the "privileged" vehicle (traveling unimpeded) must have adequate stopping sight distance to vehicles stopped at the ramp terminal.

2 Critique of Highway Design and Operational Criteria

A 1984 FHWA study provides the most recent critique of vehicle characteristics and their effect on highway design and traffic operational criteria.[33] The following discussion highlights the findings of this critique of the AASHTO intersection sight distance criteria.

a. Case I -- No Control

The analysis of case I intersection sight distance in the 1984 FHWA study focused on its sensitivity to variations in the time required to regulate speed (assumed by AASHTO to be 1 s). The current AASHTO criteria based on equation (36) was found to be insensitive to deceleration rate.[33] Since the assumption of a value for the time required to regulate speed inherently assumes a value of a vehicle characteristic--deceleration rate--a sensitivity analysis of equation (36) was performed in the 1984 FHWA study.[33]. Varying t by 0.5 s results in a 17 percent change in the required sight distance. A variation in the time required to regulate speed can represent three things: a change in the final speed reached; a change in the distance traveled while decelerating; or a change in the deceleration rate. Since the current AASHTO criteria do not include an explicit term incorporating vehicle deceleration rate, the ability to determine the criteria's sensitivity to this characteristic is limited. Because of this limitation, a new formula incorporating consideration of deceleration rate was proposed in the 1984 FHWA study:[33]

$$ISD_A = 1.47\ V_A t_{pr} + \frac{WV_A}{V_B} - \frac{d_A W^2}{2.93\ V_B^{\ 2}} \qquad (38)$$

where: ISD_A = d_B; minimum intersection sight distance for Vehicle A (ft); (see upper portion of figure 25)

V_A = design speed for vehicle A (mi/h)

t_{pr} = perception-reaction time (s) (assumed: t_{pr} = 2.0 s)

W = width of roadway on which vehicle A is traveling (ft)

V_B = design speed for vehicle B (mi/h)

d_A = deceleration rate of vehicle A (mi/h/s) (Note: if the vehicle accelerates, d_A has a negative value)

Equation (38) explicitly considers deceleration rate, but does not incorporate vehicle length and is highly dependent on perception-reaction time. Consideration of vehicle length is addressed later in the sensitivity analysis section of this report.

b. Case II -- YIELD Control

Case II intersection sight distance is based on stopping sight distance, which has been reviewed earlier in this report. Therefore, the critique of this case in the 1984 FHWA study did not provide any new insights.[33]

c. Case III -- STOP Control

Case III-A -- Crossing Maneuver: The 1984 FHWA study found case III-A to be generally insensitive to changes in the vehicle characteristic values used in current AASHTO criteria.[33] The current criteria are based on a truck with a length of 55 ft (17 m). Increasing the truck length to 70 ft (21 m), increased the required intersection sight distance by less than 10 percent. An important concern noted in the 1984 FHWA study is that the AASHTO curves for t_a (time to accelerate) were established from empirical data observed prior to 1954.[33]

Cases III-B & C -- Turning Maneuvers: As AASHTO presents these current standards, both cases lack sufficient information to derive the design curves for determining required sight distance dimensions.

3. Sensitivity Analysis

Table 44 contains a summary of the intersection sight distance parameters used in the AASHTO Green Book and the values of the vehicle-related parameters that will be varied in the subsequent sensitivity analysis. The values under the "AASHTO" heading are those used in the current criteria. They include driver-related characteristics (perception-reaction time) and vehicle-related characteristics (deceleration or acceleration time, stopping distance, and vehicle length).

The "Modifications for Truck Characteristics" in table 44 represent updated truck characteristics data. The revised acceleration rates for case I are based on the 1984 FHWA study.[33] The derivation of the stopping sight distance values for case II are discussed in section III-A of the report. Clearance times for trucks crossing intersections in case III-A are based on the Gillespie model presented in section II-D of this report.[25] Truck acceleration performance for cases III-B and III-C are based on test track data collected by Hutton.[26] Truck lengths of both 70 and 75 ft (21 and 23 m) were considered. The application of these data to derive sight distances for trucks for each intersection case is presented in the following sections.

a. Case I -- No Control

The current formula for case I intersection sight distance includes a driver characteristic in the form of the perception-reaction time. The AASHTO formula implicitly accounts for vehicle characteristics through the use of 1.0-s time to regulate speed.

Table 44. Summary of truck characteristics for intersection sight distance (ISD).

Case	AASHTO Green Book[1]: Perception-reaction time (s)	AASHTO Green Book[1]: Deceleration/acceleration time or distance	AASHTO Green Book[1]: Length of vehicle (ft)	Modifications for truck characteristics: Deceleration/acceleration Time or distance	Modifications for truck characteristics: Length of vehicle
Case I					70-ft tractor semi-trailer truck
$ISD = 1.47V_t$ $t = t_{pr} + t_r$	$t_{pr} = 2.0$	$t_r = 1.0$ s	NA	New equation $d_A = 5.5$ mi/h/s for PC $d_A = 3.63$ mi/h/s for tractor-trailer combinations	75-ft tractor semi-trailer-full trailer truck (double bottom)

Case	Perception-reaction time (s)	Speed (mi/h)	AASHTO SSD (ft)	Length of vehicle (ft)	Speed (mi/h)	Truck SSD (ft), Driver performance: Worst	Truck SSD (ft), Driver performance: Best	Length of vehicle
Case II								
$ISD = SSD = 1.47Vt_{pr} + \frac{V^2}{30(f+g)}$	$t_{pr} = 2.5$	20	125	NA	20	150	125	NA
		30	200		30	300	250	
		40	325		40	500	375	
		50	475		50	725	525	
		60	650		60	975	700	
		70	850		70	1,275	900	

Table 44. Summary of truck characteristics for intersection sight distance (ISD). (continued)

	AASHTO Green Book[1]			Modifications for truck characteristics	
Case	Perception-reaction time (s)	Deceleration/acceleration time or distance	Length of vehicle (ft)	Deceleration/acceleration Time or distance	Length of vehicle
Case III-A ISD = $1.47V(J+t_a)$	J = 2.0	t_a from AASHTO Green Book figure IX-21	19 (PC) 30 (SU) 55 (WB-50)	t_c from Gillespie equation[30]	70-ft tractor semi-trailer truck 75-ft tractor semi-trailer-full trailer truck (double bottom)
Case III-B and III-C	from AASHTO Green Book figure IX-27			t_c from Gillespie equation[30] and Hutton data[26]	70-ft tractor semi-trailer truck 75-ft tractor semi-trailer-full trailer truck (double bottom)

Note: 1 mi = 1.61 km
1 ft = 0.305 m

As discussed earlier, the 1984 FHWA study proposed an alternative equation for case I intersection sight distance that explicitly included deceleration rate (see equation (38)).[33] This equation estimates sight distances that are less than the AASHTO criteria. The equation does not adequately address case I intersection sight distance because it does not consider vehicle lengths. A tractor-trailer requires more time to cross an intersection than a passenger car because of its increased length. Therefore, a further modification of the equation is proposed to account for the length of the crossing vehicle (B) and the deceleration rate of the conflicting vehicle (A):

$$ISD_A = 1.47\ V_A\ t_{pr} + (W + L_B)\ \frac{V_A}{V_B} - \frac{d_A\ (W + L_B)^2}{2.93\ V_B^2} \qquad (39)$$

where:
- ISD_A = minimum intersection sight distance for vehicle A (ft)
- V_A = design speed for vehicle A (mi/h)
- V_B = design speed for vehicle B (mi/h)
- t_{pr} = perception-reaction time (s) (assumed: t_{pr} = 2.0 s)
- W = width of roadway on which vehicle A is traveling (ft)
- L_B = length of vehicle B (ft)
- d_A = deceleration rate of vehicle A (mi/h/s) (Note: if the vehicle accelerates, d_A has a negative value.)

Table 45 and figure 27 compare the case I intersection sight distances based on the AASHTO Green Book criteria and equation (39) for truck lengths of 70 and 75 ft (21 and 23 m). The results indicate that these longer trucks require more distance than is provided by the AASHTO criteria for vehicle B speeds up to 60 mi/h (97 km/h). The percent change in the sight distance required for vehicle A ranges from an increase of 69 percent (when V_A = 70 mi/h [113 km/h] and V_B = 20 mi/h [32 km/h]) to a decrease of 5 percent (when V_A = 20 mi/h [32 km/h] and V_B = 70 mi/h [113 km/h]).

Use of equation (39) for case I intersection sight distance is recommended because it explicitly considers both deceleration rate and vehicle length. Sight distances calculated from this formula are more sensitive to the vehicle length than to the deceleration term. The revised equation is still highly dependent on the driver perception-reaction time.

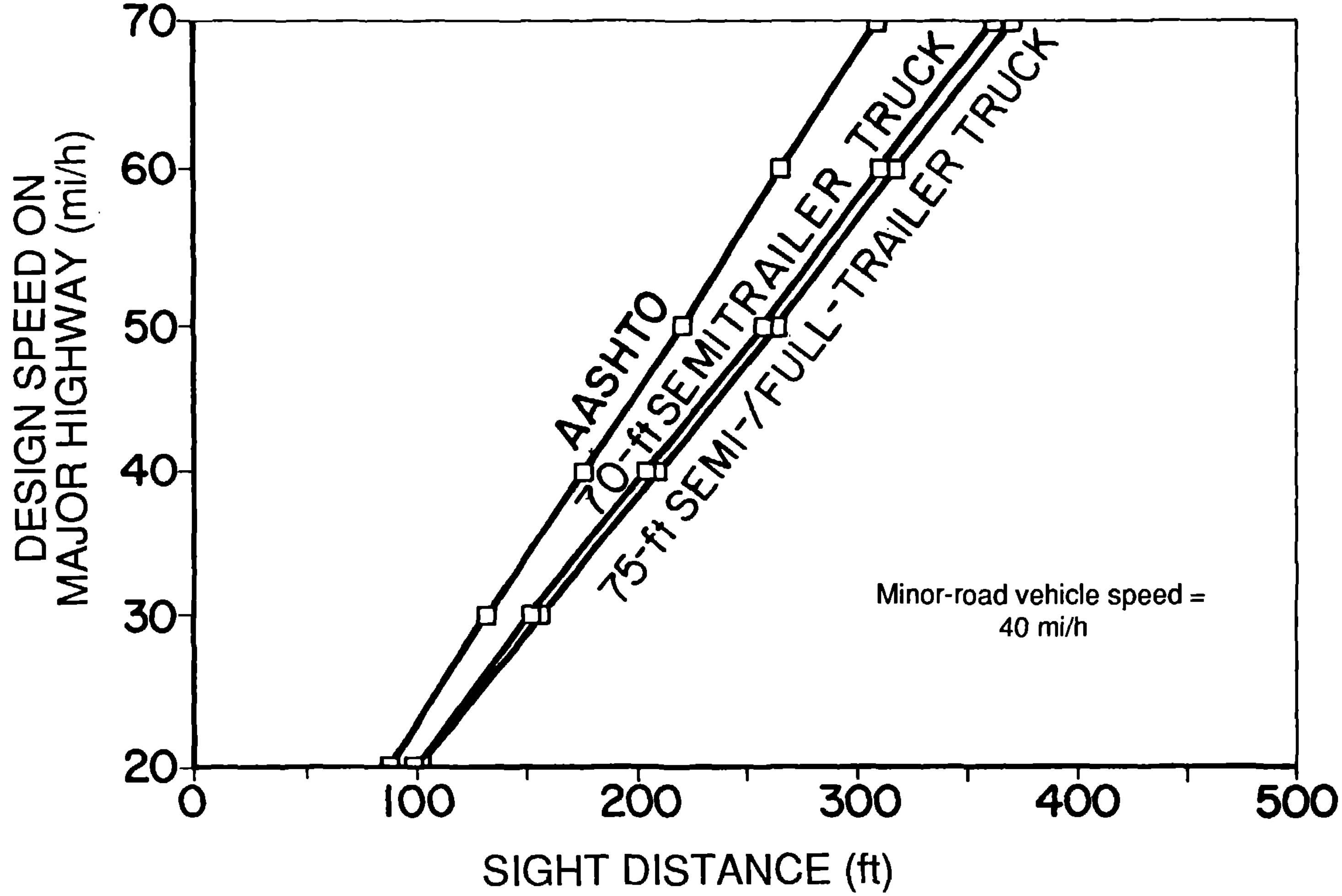
Table 45. Sensitivity analysis of case I intersection sight distance (IDS) for trucks.

Veh A speed (mi/h)	Sight dist. calc. (ft)	AASHTO Values (ft)	Speed of vehicle B (mi/h) 20 ISD (ft)	30 ISD (ft)	40 ISD (ft)	50 ISD (ft)	60 ISD (ft)	70 ISD (ft)
Veh A and veh b = 70 ft tractor semitrailer truck								
20	88	90	125	109	99	92	87	83
30	132	130	202	170	152	140	132	126
40	176	180	278	231	205	188	177	169
50	221	220	355	292	258	237	222	212
60	265	260	431	352	311	285	267	255
70	309	310	507	413	363	333	312	298
Veh A and veh B = 75-ft tractor semitrailer-full trailer truck								
20	88	90	127	111	101	94	88	85
30	132	130	206	174	155	143	134	128
40	176	180	285	236	209	192	180	172
50	221	220	364	299	263	241	226	215
60	265	260	443	361	317	290	272	259
70	309	310	522	423	371	340	318	302

See case I diagram in figure 25 for vehicle A and vehicle B sight triangles.

Assumptions: W = 24 ft (7.3 m)
d_A for 70-ft (21-m) tractor-semitrailer truck = 3.63 mi/h/s (1.62 m/s²)
d_A for 75-ft (23-m) tractor-semitrailer-full trailer truck = 3.63 mi/h/s (1.62 m/s²)
d_A values from table 33 in reference 56, 85th percentile average deceleration rate on wet pavement with an initial speed of 40 mi/h (64 km/h)

Noe: 1 mi = 1.61 km
1 ft = 0.305 m



Note: 1 mi = 1.61 km
1 ft = 0.305 m

Figure 27. Comparison of case I intersection sight distance for 70- and 75-ft (21- and 23-m) trucks to existing AASHTO criteria.

b. Case II -- YIELD Control

The case II intersection sight distance sensitivity analysis is merely an application of the AASHTO stopping sight distance formula, using the revised stopping sight distance for trucks shown in table 44. The percent increase in sight distance for the best- and worst-performance drivers in braking maneuvers in comparison to the current AASHTO criteria is shown in table 46.

c. Case III-A -- Crossing Maneuver

The current AASHTO criteria for case III-A intersection sight distance include two vehicle characteristics: (1) vehicle acceleration from a stop and (2) vehicle length. Both characteristics are used to determine the acceleration time parameter (t_a) used in the criteria. Figure IX-21 in the AASHTO Green Book provides distance versus time curves for acceleration by a passenger car, a single-unit truck, and a WB-50 truck. Vehicle length is necessary to establish the length of the hazard zone in addition to the distance from the front of the vehicle to the edge of the intersecting pavement (AASHTO assumes 10 ft or 3 m) and the width of the intersection. Table 47 shows the sight distance for an AASHTO WB-50 truck to cross a 30-ft (19 m) intersection, based on the AASHTO acceleration performance curves.

The WB-50 design vehicle is more sensitive to changes in assumed length than the other design vehicles, because: (1) a given percentage change in the length of a long vehicle is greater in absolute terms than the same percentage change in the length of a short vehicle; and (2) the lower acceleration rates of large trucks result in a longer acceleration time (t_a) over a given distance. A factor to consider in the above sensitivity analysis is that the accuracy with which the curves in figure IX-21 of the AASHTO Green Book can be read is limited. Because the curves are relatively flat, it is difficult to determine the change in t_a for small changes in distance traveled (e.g., due to small changes in vehicle length).

The acceleration time to clear a hazard zone has also been calculated using the Gillespie model (see discussion in section II-D of this report).[25] Intersection sight distances based on the Gillespie model are also shown in table 47. Use of the Gillespie model for 70- and 75-ft (21 and 23 m) trucks results in 17 and 21 percent increases, respectively, in time to cross the intersection, in comparison to the WB-50 trucks. These longer times produce a 14 percent increase in sight distance for a 70-ft (21 m) truck and a 17.5 percent increase for a 75-ft (23 m) truck. Figure 28 illustrates the results presented in table 47.

Table 46. Percent increase in case II intersection sight distance (ISD) for trucks over AASHTO criteria.

Speed (mi/h)	AASHTO SSD (ft)	Worst-performance truck driver ISD (ft)	Worst-performance truck driver Percent increase	Best-performance truck driver ISD (ft)	Best-performance truck driver Percent increase
20	125	150	20.00	125	0.00
30	200	300	50.00	250	25.00
40	325	500	53.85	375	15.38
50	475	725	52.63	525	10.53
60	650	975	50.00	700	7.69
70	850	1,275	50.00	900	5.88

Note: 1 mi = 1.61km
1 ft = 0.305 m

Table 47. Sensitivity analysis of case III-A intersection sight distance (ISD) for trucks.

Vehicle B speed (mi/h)	AASHTO ISD (ft) 55-ft truck (WB-50)	ISD (ft) based on Gillespie model[25] 70-ft tractor semitrailer truck	75-ft tractor-semitrailer-full-trailer truck
20	370	423	435
25	463	528	544
30	556	634	653
35	648	740	762
40	741	845	870
45	833	951	979
50	926	1,057	1,088
55	1,019	1,162	1,197
60	1,111	1,268	1,306
65	1,204	1,374	1,414
70	1,297	1,479	1,523

Assumptions:

Width of pavement: 30 ft (9.2 m)
Distance from edge of pavement to front of vehicle: 10 ft (3 m)
t_a determined from figure IX-21 in AASHTO Green Book
t_a = 10.6 s for a 55-ft (17-m) truck
t_c determined from Gillespie model
t_c = 12.38 s for 70-ft (21-m) truck
t_c = 12.80 s for 75-ft (23-m) truck

Note: 1 mi = 1.61 km
1 ft = 0.305 m

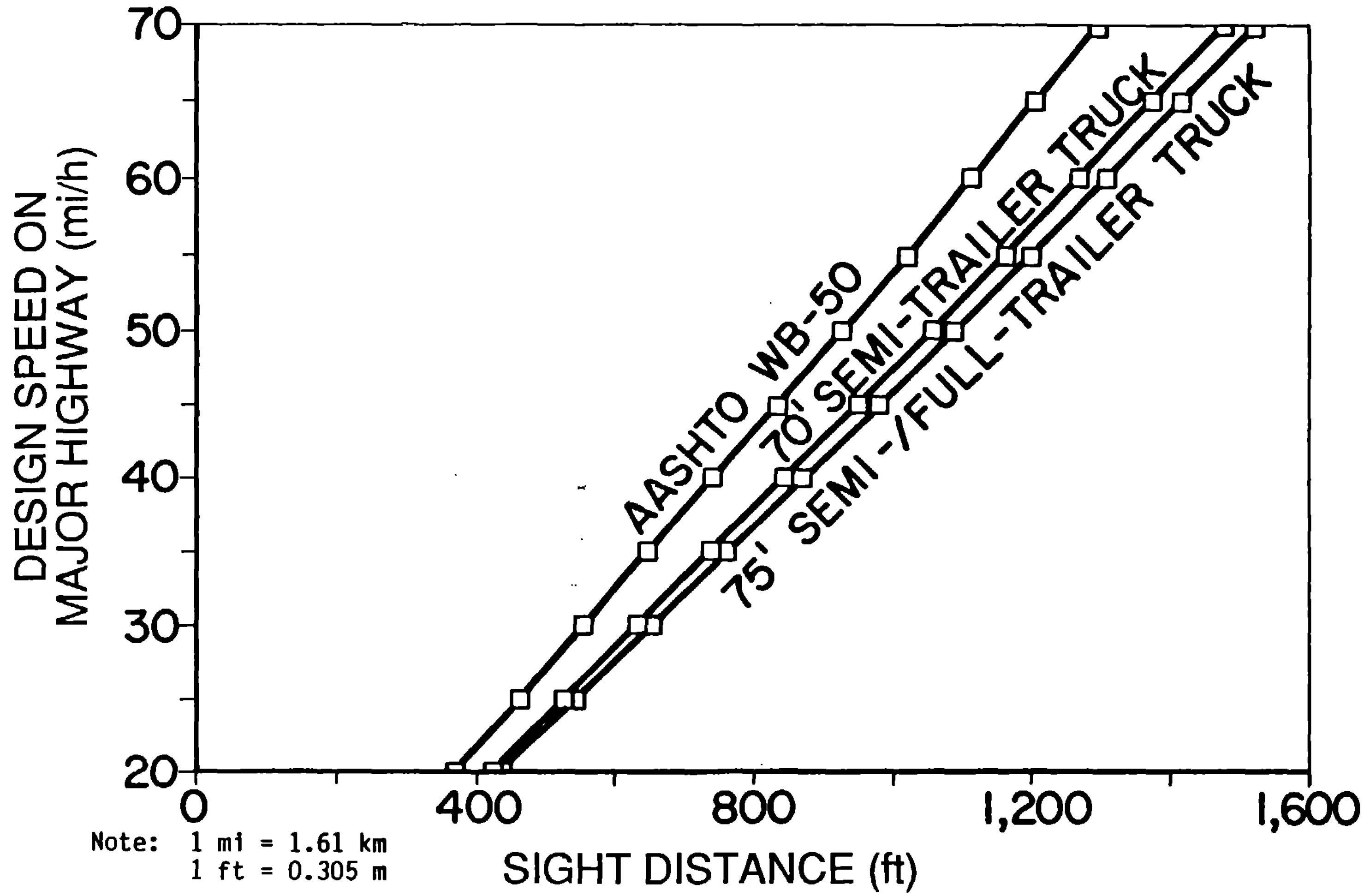


Figure 28. Comparison of case III-A intersection sight distance for 70- and 75-ft (21- and 23-m) trucks to existing AASHTO criteria.

d. Case III-B and III-C -- Turning Left or Right onto a Crossroad

AASHTO case III-B and III-C criteria include the following vehicle characteristics: acceleration from a stop, vehicle length, vehicle turning path, deceleration rate, and speed reduction. case III-B and III-C require considerable longer sight distance than case III-A because more time is needed to turn left or right and accelerate to a specified speed than is required to cross an intersecting roadway. These cases are also more complex than case III-A because the approach speed, deceleration rate, and speed reduction of the major road vehicle should be considered.

The AASHTO Green Book discussion on the B-1, B-2a & Ca, and B-2b & Cb curves (shown in AASHTO Green Book figure IX-27) lacks sufficient information to establish the values of certain parameters. Using the information that is provided and making assumptions for the missing information, the curves can be approximated. The passenger car vehicle characteristics used to reproduce the AASHTO curves can then be replaced with truck performance characteristics to estimate the truck sight distance requirements. The following discussion seeks to replicate the B-1, B-2a & Ca, and B-2b & Cb curves found in Green Book figure IX-27.

Curve B-1: This curve, as described by AASHTO, is used to establish the sight distance to be provided for a passenger car turning left onto a two-lane highway when an automobile is approaching from the left (see figure 29). The sight distance is the product of the major-road vehicle speed and the turning vehicle's acceleration time needed to clear the near lane.

$$ISD_{B-1} = 1.47\ V\ t \qquad (40)$$

$$t = t_t + J \qquad (41)$$

$$W_t = \pi\ R/2 \qquad (42)$$

where: ISD_{B-1} = sight distance along the major roadway's near lane to the left for left turns (ft) (see figure 29)

V = speed of major-road vehicle (mi/h)

t = time for a stopped minor-road vehicle to initiate the turn and clear the near lane (s)

J = sum of the perception time and the time required to actuate the clutch or actuate an automatic shift (s) (assumed: J = 2.0 s)

t_t = acceleration time required to accelerate and traverse the distance (S_t) to clear the near lane (s) (Note: data available from Green Book figure IX-22 or figure IX-21)

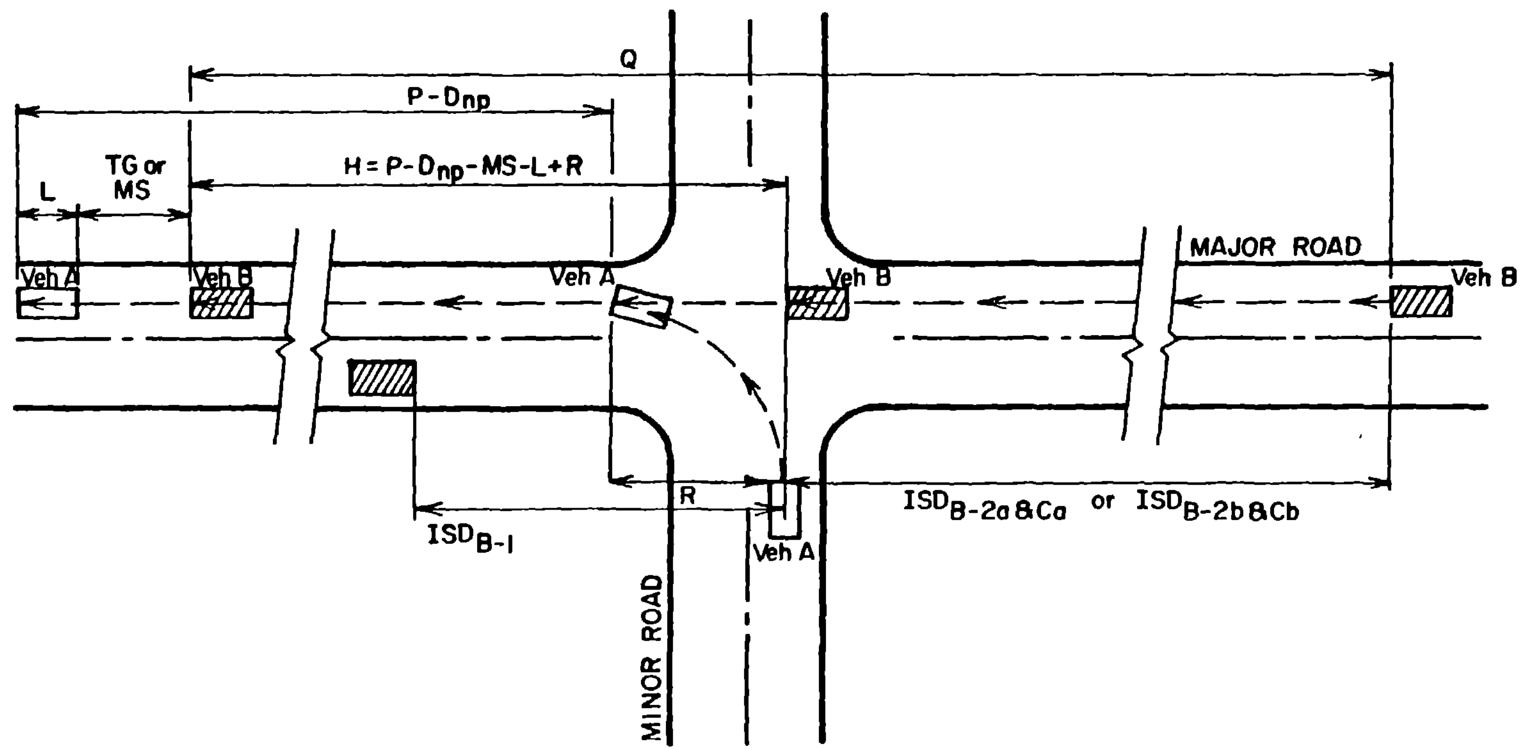


Figure 29. Distances considered in case III-B intersection sight distance criteria.

S_t = D + W_t + L, the distance that the turning vehicle must travel to clear the near lane (ft)

D = distance from the near edge of pavement to the front of a stopped vehicle (ft) (assumed: D = 10 ft or 3 m)

W_t = length of pavement traversed along path of turning vehicle (ft)

L = length of minor-road vehicle (ft)

R = radius of turn for minor-road vehicle (ft)

The values from the AASHTO B-1 curve were used to calculate acceleration times (t_t). Assuming a perception-reaction time of 2 s, the t_t value averaged 7.4 s. Using data derived from AASHTO figure IX-22, the distance traveled by a passenger car during 7.4 s is 95 ft (29 m). Assuming a vehicle length of 19 ft (6 m) and a 10-ft (3 m) distance between edge of the traveled way and the front of the vehicle, the total pavement traversed along the path of the vehicle is 66 ft (20 m). The 66-ft (20 m) vehicle path results in a 42-ft (13 m) radius. The control radius for a passenger car from Green Book table IX-20 is 40 ft (12 m). The 42-ft (13 m) radius is quite close to the value in the Green Book. Thus the acceleration time and distance values used to form the B-1 curve appear to agree with the Green Book figure IX-22.

Table 48 lists the calculated sight distances for a passenger car, an SU truck, and a WB-50 truck. Figure 30 is a plot of these sight distance values. Turning radii used were selected from AASHTO table IX-20 and are 40, 50, and 60 ft (12, 15, and 18 m). The time to clear the near lane is based upon data derived from AASHTO figure IX-22.

A sensitivity analysis can be performed on vehicle length and acceleration time to clear the intersection. The vehicle lengths used are 70 and 75 ft (21 and 23 m). The acceleration time to clear is from the Gillespie equation (see section II-D). Table 48 and figure 30 also contain the sight distances to clear the near lane for these longer trucks (CL-T70 and C-LT75). Since the Gillespie model results in less time to clear the intersection than found from figure IX-22 in the Green Book, the sight distances are shorter for the longer trucks. The 70-ft (21-m) truck sight distance (CL-T70) is 15 percent shorter and the 75-ft (23-m) truck sight distance (CL-T75) is 13 percent shorter than the sight distance for the AASHTO WB-50 truck (B-1-WB50).

Curve B-2a & Ca: This curve represents the situation in which the major road vehicle continues traveling at a constant speed equal to the highway design speed while the minor road vehicle makes its turning maneuver.

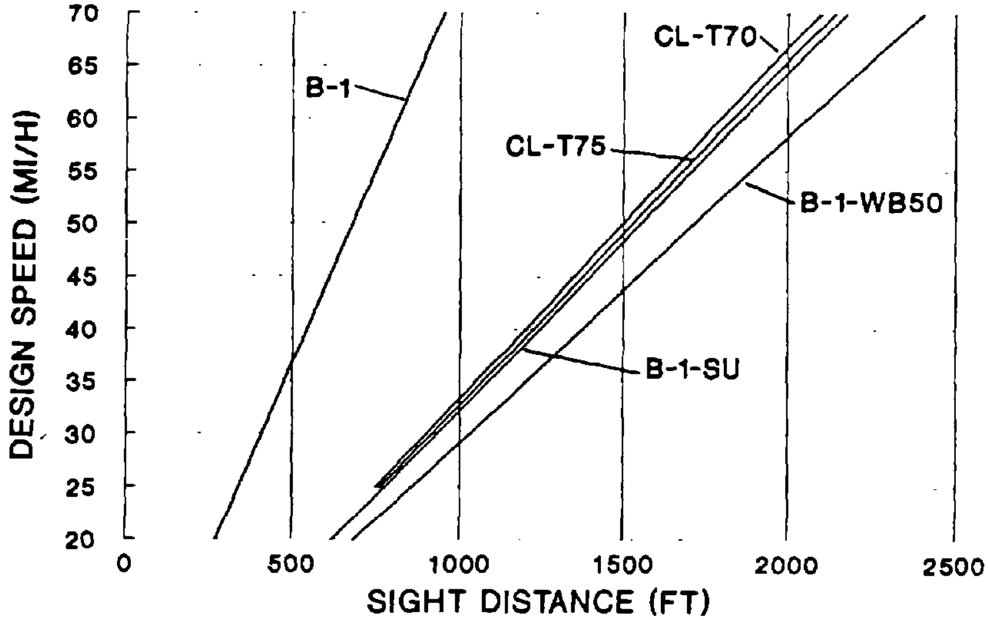
Table 48. Curve B-1 intersection sight distance values.

Speed (mi/h)	AASHTO sight distance (ft) (figure IX-27) Curve B-1	Calculated sight distance (ft) PC	SU truck B-1-SU	WB-50 truck B-1-WB-50	70-ft truck CL-T70	75-ft truck CL-T75
20	300	272	622	687	584	596
25	350	340	777	858	730	745
30	425	408	933	1,030	875	894
35	500	476	1,088	1,202	1,021	1,043
40	550	544	1,243	1,374	1,167	1,192
45	625	612	1,399	1,545	1,313	1,341
50	675	680	1,554	1,717	1,459	1,490
55	750	748	1,710	1,889	1,605	1,639
60	825	816	1,865	2,060	1,751	1,788
65	875	884	2,021	2,232	1,897	1,937
70	950	952	2,176	2,404	2,043	2,086

The following vehicle characteristics were used:

Characteristic	PC	SU	WB-50	70-ft	75-ft
Vehicle length (ft)	19	30	55	70	75
Turning radius (ft)	40	50	60	60	60
Distance to clear (ft)	92	119	159	174	179
Time to clear (s)	7.2	19.1	21.3	17.9	18.3
based on	Figure IX-22			Gillespie model[25]	

Note: 1 mi = 1.61 km
1 ft = 0.305 m



Sight distance Curve	Description
B-1	Sight distance for passenger car to clear near lane during a left turn (from Green Book figure IX-27)
B-1-SU	Sight distance for single-unit truck to clear near lane during a left turn (using Green Book truck characteristics in AASHTO procedure)
B-1-WB50	Sight distance for WB-50 truck to clear near lane during a left turn (using Green Book truck characteristics in AASHTO procedure)
CL-T70	Sight distance for 70-ft (21-m) truck to clear the near lane during a left turn (using truck characteristics from the literature in AASHTO procedure)
CL-T75	Sight distance for 75-ft (23-m) truck to clear the near lane during a left turn (using truck characteristics from the literature in AASHTO procedure

Note: 1 mi = 1.61 km
1 ft = 0.305 m

Figure 30. Intersection sight distance requirements for clearing the near lane.

Using information presented in the AASHTO Green Book, the following equations were developed to reproduce the AASHTO Curve B-2a & Ca (see dimensions defined in figure 29):

$$ISD_{B\text{-}2a\ \&\ Ca} = Q - H \quad (43)$$

$$Q = 1.47\ V\ t \quad (44)$$

$$t = t_t + J \quad (45)$$

$$H = P - D_{np} + R - TG - L \quad (46)$$

$$D_{np} = \pi * R/2 \quad (47)$$

$$TG = 1.47\ V\ t_{TG} \quad (48)$$

where: $ISD_{B\text{-}2a\ \&\ Ca}$ = sight distance along the major roadway, far lane to the right for left turns and along the near lane to the left for right turns assuming that major-road vehicles maintain constant speed during the minor-road vehicle's turning maneuver (ft) (see figure 29)

Q = distance traveled by the major-road vehicle during the minor road vehicle's turning maneuver (ft)

H = major road vehicle distance from the intersection when at assumed tailgate distance to minor road vehicle (ft)

V = speed of major-road vehicle (mi/h)

t = time for a stopped minor-road vehicle to move into traffic stream and accelerate to design speed (s)

J = sum of the perception time and the time required to actuate the clutch or actuate an automatic shift (assumed: $J = 2.0$ s)

t_t = time for minor-road vehicle to complete the turning maneuver (s) (Note: data derived from figure IX-22 in the Green Book)

P = total distance traveled by minor-road vehicle from stopped position to its location when design speed is achieved (ft) (Note: data derived from figure IX-22 in the Green Book)

D_{np} = distance minor-road vehicle traveled during the turning maneuver that is not parallel to the major highway (ft)

R = radius of turn for minor-road vehicle (ft)

TG = tailgate distance (ft)

L = length of minor-road vehicle (ft)

t_{TG} = tailgate time (s)

The AASHTO Green Book does not include a discussion of how to calculate the distance Q traversed by the major-road vehicle during the turning vehicle's maneuver. If the major-road vehicle maintains a constant speed during the turn maneuver, then the Q distance is that constant speed multiplied by the time for the minor-road vehicle to complete the turn. This time would be equal to the minor-road driver's perception-reaction time plus the time from when the vehicle began moving to when the turning vehicle has reached the same speed as the major road vehicle. A perception-reaction time value required for the turning vehicle is not mentioned in the Green Book. The perception-reaction time of 2.0 s used for the crossing maneuver (case III-A) was also assumed for the turning maneuvers.

The AASHTO Green Book does not provide information on how to derive the tailgate distance, TG. Experimenting with different values for TG to provide the closest estimate of AASHTO Curve B-2a & Ca, resulted in an estimated vehicle separation time (t_{TG}) of 1.0 s. Tailgate distance is measured from the rear of turning vehicle to the front of the oncoming vehicle. It is the product of the speed of the major-road vehicle and the 1.0-s interval.

Estimates of distance and time to accelerate were derived from Green Book figure IX-22 in 5-mi/h (8 km/h) increments. Intersection sight distance values calculated for passenger cars using the above assumptions are listed in table 49. The difference between these values and the AASHTO criteria range from a 0 to a 6 percent increase.

AASHTO states that the sight distances would be greater for trucks but does not provide specific values. The truck intersection sight distances presented in table 49 were calculated using the assumptions and equations given above. Acceleration distance and time for trucks were obtained from the truck curves in the Green Book figure IX-22. The truck intersection sight distance values thus derived are between 158 and 169 percent greater than the AASHTO B-2a & Ca curve. Figure 31 illustrates the sight distance values contained in table 49.

Table 49. Curve B-2a and Ca sight distance values.

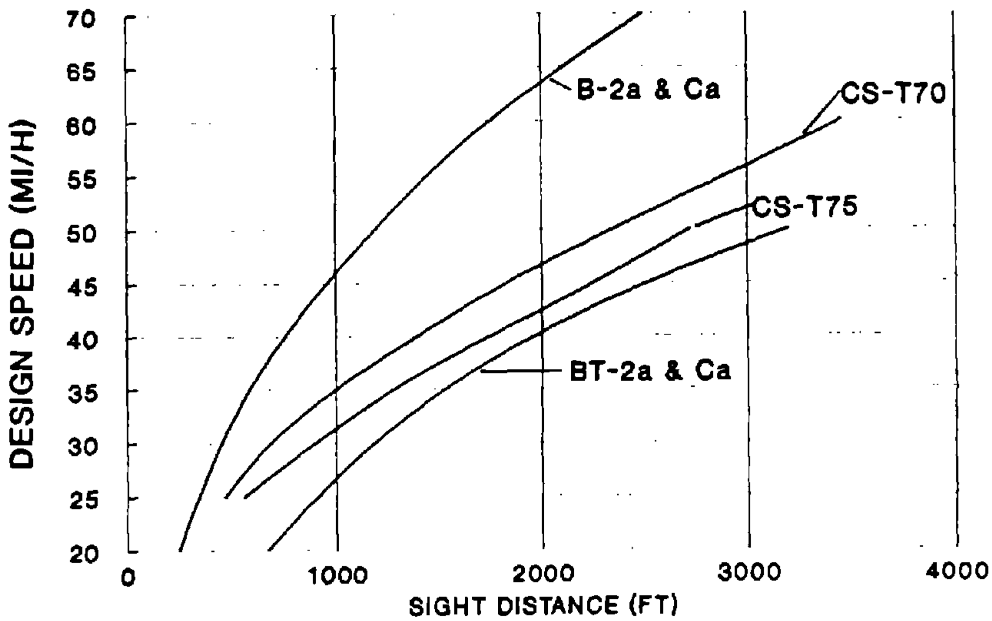
Design speed (mi/h)	AASHTO sight distance (ft) (figure IX-27) Curve B-2a & Ca	Calculated sight distance (ft) PC	Truck BT-2a & Ca	70-ft truck CS-T70	75-ft truck CS-T75
20	250	249	670	a	a
25	340	343	903	449	538
30	450	460	1,179	639	873
35	580	604	1,516	959	1,236
40	750	781	1,938	1,362	1,708
45	950	990	2,483	1,772	2,230
50	1,190	1,233	3,199	2,311	2,694
55	1,440	1,512	a	2,885	a
60	1,730	1,832	a	3,406	a
65	2,100	2,197	a	a	a
70	2,500	2,612	a	a	a

[a] Acceleration time and distance information is not available.

The following vehicle characteristics were used:

Characteristic	AASHTO PC	AASHTO Truck	70-ft truck	75-ft truck
Vehicle length (ft)	19	55	70	75
Turning radius (ft)	28	60	60	60
Acceleration from stop	Figure IX-22[1]		200 lb/hp[26]	300 lb/hp[26]

Note: 1 mi = 1.61 km
1 ft = 0.305 m
1 lb = 0.454 kg
1 hp = 746 W



Sight distance Curve	Description
B-2a & Ca	Sight distance for passenger car to turn and attain design speed (from Green Book figure IX-27)
BT-2a & Ca	Sight distance for truck to turn and attain design speed (using Green Book truck characteristics in AASHTO procedure)
CS-T70	Sight distance for 70-ft (21-m) truck to turn and attain the constant speed that the major road vehicle is driving (using truck characteristics from the literature in AASHTO procedure)
CS-T75	Sight distance for 75-ft (23-m) truck to turn and attain the constant speed that the major road vehicle is driving (using truck characteristics from the literature in AASHTO procedure)

Note: 1 mi = 1.61 km
1 ft = 0.305 m

Figure 31. Intersection sight distance curves for major-road vehicle traveling at design speed without decelerating.

Any differences in sight distance lengths between the case III-B (left turns) and III-C (right turns) situations would occur due to the different turning radii (R) between a left turn and a right turn.

Curve B-2b & Cb: This curve represents the situation in which the major-road vehicle decelerates during the turning maneuver of the minor-road vehicle, which appears to be more realistic than the previous case. The following equations were developed to reproduce the curve:

$$ISD_{B-2b\ \&\ Cb} = Q - H \qquad (49)$$

$$Q = 1.47\ V_{ds}\ t_{ds} + D_{dec} + 1.47\ V_{rs}\ t_{rs} \qquad (50)$$

$$t_{rs} = t - t_{ds} - t_{dec} \qquad (51)$$

$$t = t_t + J \qquad (52)$$

$$t^{ds} = J + t_{pr} \qquad (53)$$

$$t_{dec} = (2\ D_{dec}) / (V_{ds} + V_{rs}) \qquad (54)$$

$$H = P - D_{np} + R - TG - L \qquad (55)$$

$$Dnp = \pi\ R/2 \qquad (56)$$

$$TG = 1.47\ V_{rs}\ t_{TG} \qquad (57)$$

where:

$ISD_{B-2b\ \&\ Cb}$ = sight distance along the major roadway's far lane to the right for left turns and along the near lane to the left for right turns assuming that a major-road vehicle reduces speed from design speed to running speed during minor-road vehicle's turning maneuver (ft)

Q = distance traveled by the major-road vehicle during the minor-road vehicle's turning maneuver (ft)

V_{ds} = design speed for the major-road vehicle (mi/h)

t_{ds} = time major-road vehicle is at design speed during turning maneuver (s);

D_{dec} = distance major-road vehicle traversed during deceleration (ft)

t_{dec} = time major-road vehicle is decelerating (s)

V_{rs} = running speed of major-road vehicle (mi/h)

t_{rs} = time major-road vehicle is at running speed during turning maneuver (s)

H = major-road vehicle's distance from intersection when at assumed tailgate distance to minor-road vehicle (ft)

t = time for a stopped minor-road vehicle to move into traffic stream and accelerate to design speed (s)

J = sum of the perception time and the time required to actuate the clutch or actuate an automatic shift (s); (assumed: $J = 2.0$ s)

t_t = acceleration time for the minor-road vehicle to complete the turning maneuver (s) (data derived from Green Book figure IX-22)

t_{pr} = perception-reaction time for the major-road driver (s); (assumed: $t_{pr} = 2.0$ s)

P = total distance traveled by minor-road vehicle from stopped position to location when design speed is achieved (ft); (data derived from Green Book figure IX-22)

D_{np} = distance minor-road vehicle traveled during the turning maneuver that is not parallel to major highway (ft)

R = radius of turn for minor-road vehicle (ft)

TG = tailgate distance (ft)

L = length of minor-road vehicle (ft)

t_{TG} = tailgate time (s) (assumed: $t_{TG} = 1.0$ s)

The calculation for the distance traveled by the major-road vehicle during the turning maneuver is more complex than the previous situation. The Q distance is comprised of three segments: (1) distance traveled at design speed; (2) distance traveled while decelerating from design speed to running speed; and (3) distance traveled at running speed. The time at the design speed was assumed to be equal to the minor-road driver's perception-reaction time (J) and the major-road driver's perception-reaction time (t_{pr}). This assumes that the major-road driver begins to decelerate when the initiation of the minor-road vehicle's turn maneuver is perceived.

The distance to decelerate was derived from Green Book figure II-13 for speed reductions to a minimum of 50 mi/h (80 km/h). Reductions to 55 mi/h (89 km/h) or more can be determined using a comfortable deceleration rate of 3.3 mi/h/s as discussed in the Transportation and Traffic Engineering Handbook.[71] The time to decelerate can be calculated from the distance to decelerate using equation (54). The time spent at running speed can then be calculated by subtracting the time at design speed and time to decelerate from the turning maneuver time.

The distance traveled by the minor-road vehicle for this situation is similar to the constant speed situation except that the vehicle is accelerating to the running speed instead of the design speed. The tailgate distance is based on running speed.

Trial and error was used to estimate the speed reduction of the major-road vehicle used by AASHTO. The findings based on the above assumptions predicted the values along curve B-2b & Cb within 8 percent. The speed reductions in 5 mi/h (8 km/h) increments that provided the best predictions of the AASHTO curves were:

- No speed reduction for design speeds less than 30 mi/h (48 km/h)
- Five mi/h (8 km/h) speed reduction for design speeds between 30 and 65 mi/h (48 and 105 km/h)
- Ten mi/h (16 km/h) speed reduction for design speed of 70 mi/h (113 km/h)

Table 50 presents the results using the above assumptions.

The sight distance for trucks can now be calculated using the above equations and the truck acceleration data derived from Green Book figure IX-22. These sight distance values (BT-2b & Cb) are listed in table 50 and the resulting curves are illustrated in figure 32. The calculated truck sight distances are between 131 and 178 percent greater than the values given by AASHTO intersection sight distance B-2b & Cb curve.

Sensitivity Analysis for Curves B-2a & Ca and B-2b & Cb: With the step-by-step methodology defined for the derivation of the Green Book B-2a & Ca and B-2b & Cb curves, sensitivity analyses were performed on the following vehicle characteristics: acceleration time and distance, and vehicle length. These analyses focused on the differences resulting from use of a truck rather than a passenger car as the minor-road vehicle. The major-road vehicle was assumed to be a passenger car; therefore, the deceleration rate and speed reduction did not change.

AASHTO defines tailgate distance as the minimum distance between the rear bumper of the turning vehicle and the front bumper of the major-road vehicle. The term tailgate seems to imply inappropriate driving behavior during a turning maneuver; the minimum distance is not necessarily an improper action by the major-road driver. Therefore, the term minimum separation has been used in the following analysis.

The sight distances for Curves B-2a & Ca and B-2b & Cb using modified truck characteristics (CS for major-road vehicle maintaining constant speed and RS for major-road vehicle reducing speed) are given in tables 49 and 50. The findings are also illustrated in figures 31 and 32. When the minor-road

Table 50. Curve B-2b and Cb sight distance values.

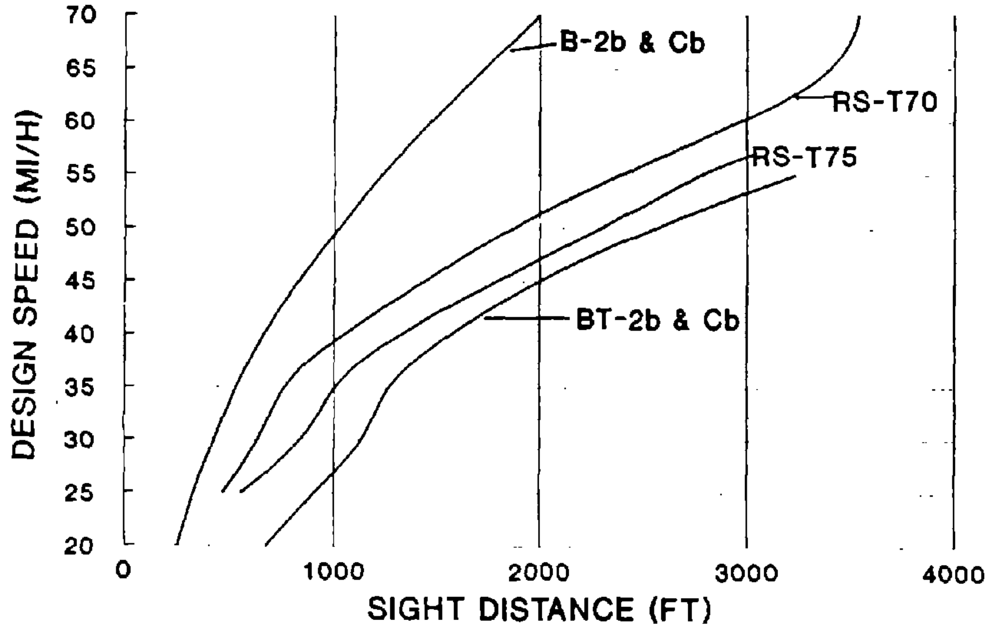
Design speed (mi/h)	AASHTO sight distance (ft) (figure IX-27) Curve B-2b & Cb	Calculated sight distance (ft)			
		PC	Truck BT-2b & Cb	70-ft truck RS-T70	75-ft truck RS-T75
20	250	249	670	a	a
25	325	343	903	449	538
30	425	460	1,179	639	873
35	525	494	1,213	673	907
40	660	638	1,549	993	1,270
45	825	814	1,971	1,395	1,741
50	1,025	1,023	2,516	1,804	2,263
55	1,225	1,266	3,232	2,343	2,727
60	1,475	1,545	a	2,918	a
65	1,725	1,865	a	3,439	a
70	2,000	1,906	a	3,480	a

[a] Acceleration time and distance information is not available.

The following vehicle characteristics were used:

Characteristic	AASHTO PC	AASHTO Truck	70-ft truck	75-ft truck
Vehicle length (ft)	19	55	70	75
Turning radius (ft)	28	60	60	60
Acceleration from stop	Figure IX-22[1]		200 lb/hp[26]	300 lb/hp[26]

Note: 1 mi = 1.61 km
1 ft = 0.305 m
1 lb = 0.454 kg
1 hp = 746 W



Sight distance Curve	Description
B-2b & Cb	Sight distance for passenger car to turn and attain average running speed (from Green Book figure IX-27)
BT-2b & Cb	Sight distance for truck to turn and attain average running speed (using Green Book truck characteristics in AASHTO procedure)
RS-T70	Sight distance for 70-ft (21-m) truck tot urn and attain the reduced speed that the major-road vehicle is driving (using truck characteristics from the literature in AASHTO procedure)
RS-T75	Sight distance for 75-ft (23-m) truck to turn and attain the reduced speed that the major-road vehicle is driving (using truck characteristics from the literature in AASHTO procedure)

Note: 1 mi = 1.61 km
1 ft = 0.305 m

Figure 32. Intersection sight distance curves for deceleration by major-road vehicle from design speed to average running speed.

vehicle is assumed to be a 70-ft (21-m) truck with a weight-to-power ratio of 200 lb/hp (0.12 kg/W) rather than a passenger car, the resulting sight distance is between 32 and 100 percent greater than the AASHTO values for passenger cars. If a 75-ft (23-m) truck is used, the required sight distance is between 58 and 135 percent greater than the value for a passenger car.

4. Field Data Collection

Pilot field studies were conducted at three intersections to test a data collection methodology in order to evaluate case III-B and -C intersection sight distance requirements for trucks at STOP-controlled "T" intersections. The objectives of the field data collection efforts were to:

- Develop a data collection methodology to determine acceleration, deceleration, speed reduction, minimum separation, and gap acceptance characteristics.

- Perform a pilot test of the methodology via data collection at three intersections.

- Compare the field data to AASHTO Green Book values.

Specific details of the study efforts and results are included in appendix E of this report. The pilot field studies were considered adequate to develop data collection techniques to guide future efforts. A larger-scale study is needed to fully develop the gap acceptance concept for a broader range of vehicle types, driver types, intersection geometrics, and approach speeds. Such a study would also provide the necessary information to improve practical application of the Green Book ISD criteria.

a. Summary of Field Study Findings

The findings from the pilot field study are summarized in the following series of tables:

Table 51 -- time gaps accepted (s) at 50th and 85th percentile probabilities. These gaps were determined using a logit model. The table is arranged by intersection, maneuver type, and truck type.

Table 52 -- acceleration rates (mi/h/s) for the predominant truck type for both left and right turns at one intersection and for right turns at the remaining two intersections.

Table 53 -- deceleration rates (mi/h/s) and speed reductions (mi/h) for major-road vehicles impeded by five-axle trucks turning right at two intersections.

Table 54 -- minimum separation times and distances determined from the limited data for two intersections. These findings should be used carefully due to a small sample size.

Table 51. Time gaps accepted from field data.

Intersection	Turn maneuver	Truck type	Time gap (s) 50th percentile	85th percentile
Central Valley Asphalt	Left	less-than-five-axle	11.16	13.89
Central valley Asphalt	Right	less-than-five-axle	13.17	15.87
Truck Stop 64	Right	five-axle	12.43	14.78
Trindle and Railroad	Left	five-axle	8.27	9.84
Trindle and Railroad	Right	five-axle	8.52	10.06
Trindle and Railroad	Right	less-than-five-axle	7.25	8.87

Table 52. Acceleration rates from field data.

Intersection	Turn maneuver	Truck type (no. of axles)	Distance (ft)	Acceleration rate (mi/h/s) 50th percentile	85th percentile
Central Valley Asphalt	Left	4	0-290	1.27	1.58
Central Valley Asphalt	Right	4	0-490	1.04	1.21
Truck Stop 64	Right	5	0-350	0.80	1.20
Trindle and Railroad	Right	5	0-510	1.37	1.74

Note: 1 ft = 0.305 m
1 mi = 1.61 km

Table 53. Deceleration rates and speed reductions for major road vehicles impeded by right turns by five-axle trucks.

	Cumulative probability	
	50th percentile	85th percentile
Deceleration rates	3.67 mi/h/s	5.85 mi/h/s
Speed reductions	21.2 mi/h	38.1 mi/h

Note: 1 mi = 1.61 km

Table 54. Minimum separation times and distances.

Intersection	Headway time (s)	Minimum separation distance (ft)
Truck Stop 64	1.00	25
	0.63	25
	2.17	25
	1.33	25
	1.07	25
	2.38	25
	0.86	25
Trindle and Railroad	5.01	109
	4.38	91
	4.13	143
	4.80	88
	4.53	57
	5.24	75

Note: 1 ft = 0.305 m

b. Discussion of Results

One objective of the pilot study was to compare the resulting field data with the Green Book intersection sight distance policy. Figures 33 and 34 contains the intersection sight distance curves developed from the following sources:

- AASHTO sight distance model using Green Book estimates of truck performance (BT-2a & Ca and BT-2b & Cb curves).
- AASHTO sight distance model using truck performance estimates from the literature (Curves SSD-TW, SSD-TB, CS-T70, CS-T75, RS-T70, RS-T75).
- Intersection sight distance requirements based on gap acceptance methodology (G-7, G-10, and G-15 curves).

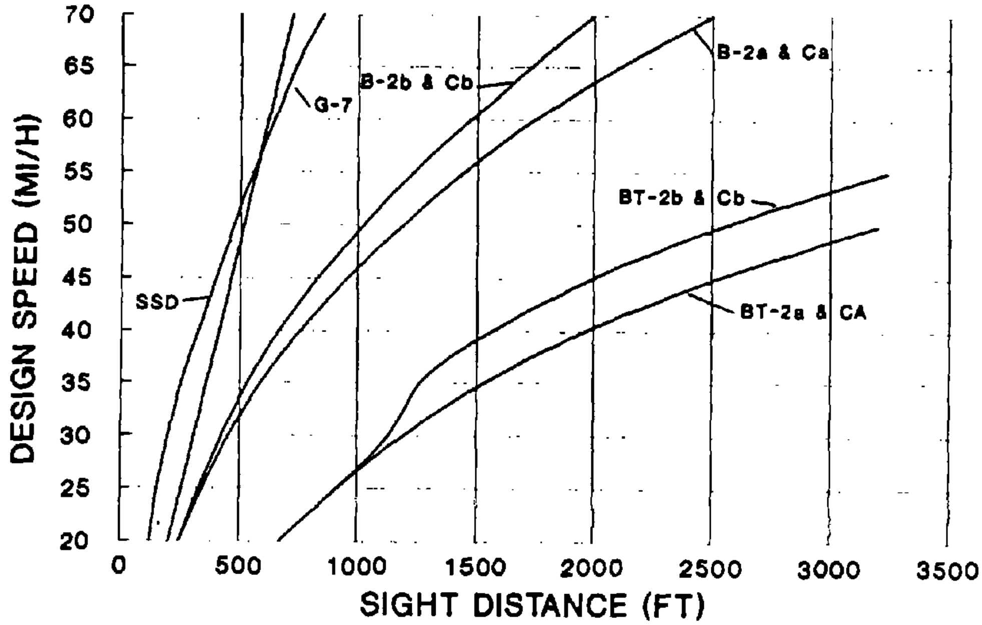
The following discussion briefly describes the basis for each curve. These curves are explained more fully in appendix E in volume II of this report.

Curve B-2a & Ca in figure 33 represents the safe sight distance for a passenger car to turn left or right onto a two-lane highway and attain design speed without being overtaken by a vehicle approaching from the right and traveling at a constant speed equal to the design speed. Curve B-2b & Cb represents the safe sight distance for a passenger car to turn left or right onto a two-lane highway and attain average running speed without being overtaken by vehicle approaching from the right and reducing its speed from the design speed to the average running speed.

The Green Book indicates that sight distance for trucks will be considerably longer than for passenger vehicles but does not provide the curves or a clearly defined method to determine the sight distance needed by a truck. The truck acceleration curves found in Green Book figure IX-22 were used together with the existing AASHTO model for cases III-B and -C to derive the intersection sight distance curves for trucks (BT-2b & Cb and BT-2a & Ca) shown in figure 33.

The SSD curve in 33 represents the stopping sight distance for a passenger car on a wet pavement. The values are derived from table III-1 in the Green Book.

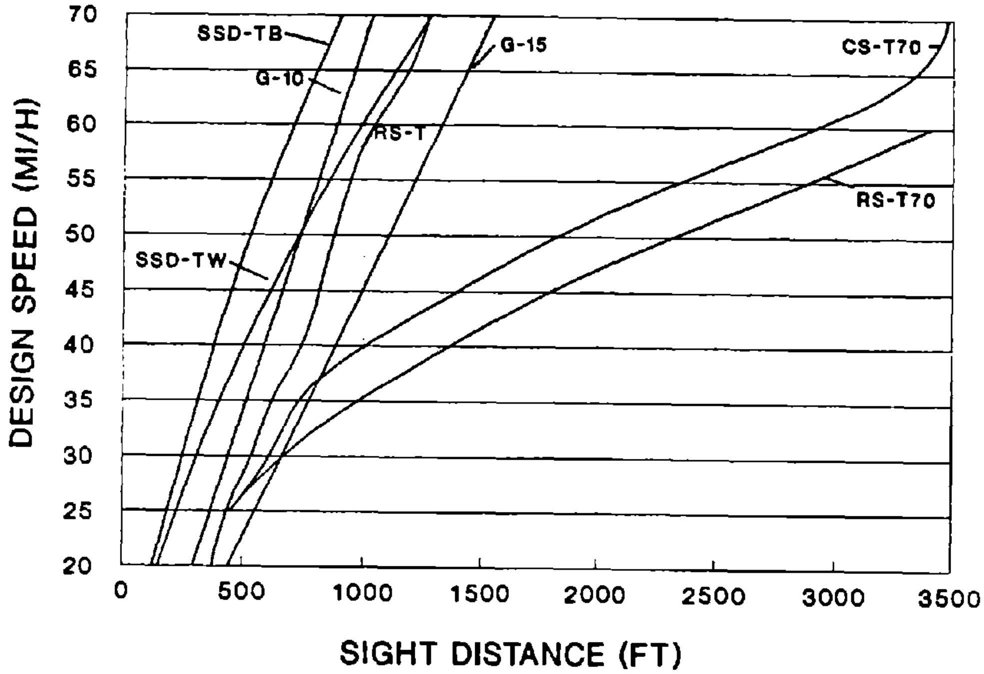
The stopping sight distance values for trucks in the lower portion of figure 34 represent controlled braking by an empty truck on a poor, wet road with relatively good radial tires for a worst-performance driver (SSD-TW) and a best-performance driver (SSD-TB). The constant speed (CS) curves represent the case in which the major-road vehicle maintains a constant speed equal to the highway design speed, while the reduced speed (RS) curves represent the case in which the major-road vehicle reduces speed from the design speed to the average running speed.



Sight distance Curve	Description
G-7	Sight distance from gap acceptance procedure for 7-s gap
SSD	Stopping sight distance for a passenger car to turn and attain design speed (from Green Book figure IX-27)
B-2a & Ca	Sight distance for a passenger car to turn and attain design speed (from Green Book figure IX-27)
BT-2a & Ca	Sight distance for a truck to turn and attain design speed (using Green Book truck characteristics in AASHTO procedure)
B-2b & Cb	Sight distance for a passenger car to turn and attain average running speed (from Green Book figure IX-27)
BT-2b & Cb	Sight distance for a truck to turn and attain average running speed (using Green Book truck characteristics in AASHTO procedure)

Note: 1 mi = 1.61 km
1 ft = 0.305 m

Figure 33. Intersection sight distance curves.



Sight distance Curve	Description
G-10	Sight distance from gap acceptance procedure for 10-s gap
G-15	Sight distance from gap acceptance procedure for 15-s gap
SSD-TB	Stopping sight distance for truck with conventional brake system and best-performance truck driver (from table 23)
SSD-TW	Stopping sight distance for truck with conventional brake system and worst-performance truck driver (from table 23)
RS-T	Sight distance for a truck from this field study to turn and attain the reduced speed that the major-road vehicle is traveling (using findings from field data in AASHTO procedure)
CS-T70	Sight distance for 70-ft (21-m) truck to turn and attain the constant speed that the major-road vehicle is traveling (using truck characteristics from the literature in AASHTO procedure)
RS-T70	Sight distance for 70-ft (21-m) truck to turn and attain the reduced speed that the major-road vehicle is traveling (using truck characteristics from the literature in AASHTO procedure)

Note: 1 mi =1.61 km
1 ft = 0.305 m

Figure 34. Intersection sight distance curves for present study.

The preliminary results of the pilot study can be used to illustrate the implications of the field data that were collected when used as parameters in the current AASHTO intersection sight distance model for cases III-B and -C. These results should be used with caution since they are based on a limited pilot study. Furthermore, the plotted results are based only on data for five-axle trucks turning right. The RS-T curve in figure 34 represents the results from using the field data in the existing AASHTO model.

One objective of the field study was to test a new concept for intersection sight distance based on gap acceptance. Under this concept, field studies were conducted to determine the lengths of gaps that were safely accepted by turning trucks, and intersection sight distance criteria would be established to assure that the available sight distance was at least equal to that acceptable gap length. Figure 34 illustrates intersection sight distance curves based on acceptable gaps of 7, 10, and 15 s. The 7-s gap acceptance criterion is suggested by the Green Book. The 10- and 15-s gaps were selected based on the 85th percentile gap acceptance probabilities from two intersections--a high-volume and a low-volume intersection, respectively.

c. Comparison of ISD Curves

When the curves derived directly from ISD criteria and vehicle characteristics given in the Green Book are compared (see figure 33), three general groupings result: (1) ISD criteria based on the AASHTO model and truck acceleration values from Green Book figure IX-22 curves (BT-2a & Ca and BT-2b & Cb); (2) AASHTO ISD criteria for passenger cars as given in the Green Book (B-2a & Ca and B-2b & Cb); and (3) the curves for AASHTO SSD and the ISD criteria based on a 7-s gap.

The ISD curves from the pilot study fall into two groups (see figure 34). One group consists of the ISD criterion based on the AASHTO model and truck performance acceleration values from the literature. The group includes both the major-road vehicle at CS and RS conditions. The other group consists of the remaining curves (G-10, G-15, SSD-TW, SSD-TB, and RS-T). The curve based on findings from the field data collection (RS-T) is between the 10-s gap curve and the 15-s gap curve. The SSD values for both best and worst performance truck drivers (SSD-TB and SSD-TW) are less than the values based on the field studies.

5. Summary of Findings

A revised model developed in this study indicates that intersection sight distance for case I (no control) is quite sensitive to vehicle length, which is not considered in the current AASHTO criteria. Sensitivity analysis results indicate that trucks require greater case I intersection sight distance than the current AASHTO criteria for all approach speeds considered and for all crossing vehicle speeds up to 60 mi/h (97 km/h).

The intersection sight distance procedure for case II (YIELD control) is an application of the stopping sight distance formula. Stopping sight distance requirements for trucks depend on driver braking performance. The best performance driver requires up to a 25 percent increase in additional intersection sight distance; the worst performance driver needs a 20 to 54 percent increase in required sight distance. The increased driver eye height for trucks, compared to passenger cars, may offset part of this increase in required sight distance where sight distance is limited by a vertical obstruction.

A sensitivity analysis found that 70- and 75-ft (21- and 23-m) combination trucks require substantially longer intersection sight distance than an AASHTO WB-50 truck for case III-A (STOP control, crossing maneuver). In particular, intersection clearance times based on the Gillespie model indicate that a 70-ft (21 m) truck requires 14 percent more sight distance than an AASHTO WB-50 truck, and a 75-ft (23-m) truck requires 17.5 percent more sight distance.

The sensitivity analysis also found that the selected trucks would require substantially more intersection sight distance than passenger cars for cases III-B and III-C (STOP control, turning left or right onto a cross road). The additional sight distance requirements of trucks vary as a function of weight-to-power ratio. A 200 lb/hp (0.12 kg/W), 70-ft (21 m) truck requires between 32 and 100 percent additional sight distance compared to a passenger car, and a 300 lb/hp (0.18 kg/W), 75-ft (23 m) truck requires between 58 and 135 percent additional sight distance.

6. Recommended Revisions to Design and Operational Criteria

No specific revisions to existing AASHTO intersection sight distance criteria are recommended at this time. The analyses based on extending the current AASHTO intersection sight distance model to determine sight distance requirements resulted in increased sight distance requirements for each intersection case. For case I, II, and III-A, the largest additional truck sight distance requirement ranged from approximately 125 to 450 ft (38 to 137 m). Cases III-B and -C can require more than 3,000 ft (900 m) of sight distance in some cases.

It is clear from operational experience that sight distances as long as 3,000 ft (900 m) are not necessary for safe operations at intersections, even where large trucks are present. Very few intersections have such long sight distances available, and it is unlikely that either passenger car or truck drivers could accurately judge the location and speed of an oncoming vehicle at a distance of 3,000 ft (900 m). Rather, this result indicates that the current AASHTO model for cases III-B and III-C for truck intersection sight distance, on which this analysis is based, is unrealistic. In particular, it is unrealistic to assume that potentially conflicting vehicles on the main road will make only minor adjustments in speed if a truck from the side road makes a left or right turn.

The authors see a need to revise or replace the AASHTO model for cases III-B and III-C intersection sight distance especially for trucks. The pilot field studies reported above are a first step toward acquisition of the data needed either to revise the AASHTO model to include realistic deceleration by the major-road vehicle or to replace the AASHTO model with an alternative model based on gap acceptance.

No cost-effectiveness analyses of candidate revisions to design criteria were conducted because no specific revisions to the intersection sight distance criteria have been recommended. However, a supplementary cost-effectiveness analysis was conducted to determine whether there could be economic justification for clearing of the sight triangle illustrated in the upper portion of figure 25 to provide additional case II sight distance to accommodate trucks at rural intersections with YIELD control.

Case II intersection sight distance is equivalent to stopping sight distance, so the alternative design criteria to be evaluated are those in table 24. This analysis addresses a situation that is a departure from current AASHTO criteria, which do not require the full stopping sight distance appropriate for the design speed of a YIELD-controlled approach to be provided. Rather, current criteria encourage the use of advisory speed limit signing when the corner sight triangle does not provide the full stopping sight distance for the design speed of the approach.

The cost of providing additional case II sight distance for trucks is highly variable and depends on the specific sight obstructions that are present in each quadrant of the interchange. Expanding the sight triangle could be as simple as clearing brush and as complicated as removing a structure. Table 55 shows the number of additional acres that would need to be cleared per quadrant of an intersection to provide stopping sight distance for trucks, as a function of the major- and minor-road design speeds. The table also shows the cost to clear all four quadrants of an intersection, based on the assumption that brush and trees up to 10-in (25-cm) diameter can be cleared for $2,150/acre ($7,000/ha). If structures were present, the cost to purchase and remove those structures could be $200,000 per quadrant or higher. Note that the costs in the table and the results of the subsequent analysis assume that the sight triangle to provide stopping sight distance to meet current AASHTO criteria is already clear, which at many locations is not necessarily the case.

Table 56 presents the percentage reductions in truck accidents that would be required to make the provision of additional sight distance cost effective. These results are presented as a function of three factors: (1) the traffic volume entering the intersection (veh/day); (2) the percent trucks in the traffic stream; and (3) the cost to clear all four quadrants of the intersection. The clearing costs range from $1,000 to $500,000 per intersection to cover the entire range of conditions that could be encountered in the field. The table shows that very inexpensive clearing operations (e.g., $1,000 per intersection) could be very cost effective, even at relatively low volumes. On the other hand, very expensive clearing operations (e.g., removing structures or embankment costing $100,000 or more) are almost never cost effective.

Table 55. Additional area of clear sight triangle and clearing costs to provide case II sight distance for trucks at rural intersections.

Minor road design speed (mi/h)	Major road design speed (mi/h) 20	30	40	50	60	70
	Additional area to be cleared per quadrant (acres)[a]					
20	0.03	0.08	0.17	0.24	0.32	0.42
30		0.26	0.53	0.76	1.01	1.34
40			1.09	1.56	2.08	2.75
50				2.25	2.99	3.95
60					3.97	5.23
70						6.93
	Additional clearing cost for four-quadrant intersection ($)[b]					
20	$338	$ 915	$ 1,821	$ 2,636	$ 3,529	$ 4,672
30		2,907	5,786	8,336	11,124	14,713
40			11,970	17,203	22,910	30,287
50				24,798	32,872	43,406
60					43,624	57,551
70						76,203

[a] Based on difference between passenger car and truck sight distances given in table 24.

[b] Based on clearing cost of $2,750/acre ($6,960/ha).

Note: 1 mi = 1.61 km
1 acre = 0.395 ha

Table 56. Minimum percent reduction in truck accidents at rural intersections required for cost effectiveness of providing larger clear sight triangles.

Average daily traffic volume (veh/day)	Additional clearing cost per intersection ($)					
	$1,000	$5,000	$10,000	$50,000	$100,000	$500,000
1% Trucks						
2,000	53.0	265.1	530.2	2,651.1	5,302.2	26,511.1
4,000	26.5	132.6	265.1	1,325.6	2,651.1	13,255.5
6,000	17.7	88.4	176.7	883.7	1,767.4	8,837.0
8,000	13.3	66.3	132.6	662.8	1,325.6	6,627.8
10,000	10.6	53.0	106.0	530.2	1,060.4	5,302.2
5% Trucks						
2,000	10.8	53.9	107.9	539.3	1,078.7	5,393.3
4,000	5.4	27.0	53.9	269.7	539.3	2,696.7
6,000	3.6	18.0	36.0	179.8	359.6	1,797.8
8,000	2.7	13.5	27.0	134.8	269.7	1,348.3
10,000	2.2	10.8	21.6	107.9	215.7	1,078.7
10% Trucks						
2,000	5.5	27.6	55.1	275.6	551.2	2,755.9
4,000	2.8	13.8	27.6	137.8	275.6	1,377.9
6,000	1.8	9.2	18.4	91.9	183.7	918.6
8,000	1.4	6.9	13.8	68.9	137.8	689.0
10,000	1.1	5.5	11.0	55.1	110.2	551.2
20% Trucks						
2,000	2.9	14.4	28.8	144.1	288.2	1,441.2
4,000	1.4	7.2	14.4	72.1	144.1	720.6
6,000	1.0	4.8	9.6	48.0	96.1	480.4
8,000	0.7	3.6	7.2	36.0	72.1	360.3
10,000	0.6	2.9	5.8	28.8	57.6	288.2
30% Trucks						
2,000	2.0	10.1	20.1	100.7	201.4	1,007.1
4,000	1.0	5.0	10.1	50.4	100.7	503.5
6,000	0.7	3.4	6.7	33.6	67.1	335.7
8,000	0.5	2.5	5.0	25.2	50.4	251.8
10,000	0.4	2.0	4.0	20.1	40.3	201.4

E. Intersection and Channelization Geometrics

1. Current Highway Design and Operational Criteria

The horizontal geometry of intersection turning roadways is controlled by the paths of the outer front wheel, the inner rear wheel, and front overhang of a design truck. The AASHTO Green Book establishes the minimum turning path for design trucks based on the boundaries of the outer trace of the front overhang and the sharpest turning radius of the right inner rear wheel. Minimum turning radius is defined as the path of the outer front wheel, following a circular arc, at a speed of less than 10 mi/h (16 km/h), and is limited by the vehicle steering mechanism. Minimum inside radius is the path traced by the right rear wheel. The principal vehicle characteristics that govern the horizontal geometry of an intersection are its overall width, effective wheelbase length, and minimum turning radius. Current AASHTO criteria are based on the three design vehicles shown in table 57.

Table 57. Width, wheelbase, and turning radii of AASHTO design vehicles.

	Width (ft)	Wheelbase (ft)	Minimum turning radius (ft)
WB-40	8.5	40	40
WB-50	8.5	50	45
WB-60	8.5	60	45

Because a truck has a long wheelbase, its rear wheels do not follow the same path as its front wheels during a turn. The differences in these paths are referred to as "offtracking." Offtracking values vary directly with the wheelbase of a unit and inversely with the radius of turn. "Swept path width," the difference in paths of the outside front tractor tire and the inside rear trailer tire, is a more appropriate parameter for design consideration. Swept path width determinations delineate the boundaries of "critical space" occupied by the vehicle negotiating its turn. The definitions of terms used here in relation to truck offtracking and swept path widths are illustrated in figure 13.

a. Minimum Design for Sharpest Turns

In the design of the edge of pavement for the minimum path of a given design vehicle, the AASHTO Green Book assumes that the vehicle is properly positioned within the traffic lane at the beginning and end of the turn (i.e., 2 ft [0.6 m] from the edge of pavement on the tangents approaching and leaving the intersection curve). The Green Book provides tables and figures that demonstrate the minimum design for particular design vehicles and various intersection geometrics. AASHTO notes that it may not be practical to fit simple circular arcs to their minimum design paths. Asymmetrical three centered curves or tapered configurations may be the preferred design.

b. Channelized Islands

Channelization serves to control and direct traffic movement. The AASHTO policy offers guidance on the purpose of intersection channelization and provides details of island design considerations. Design vehicle characteristics are implicitly addressed through the pertinent selection of turning path or roadway radii.

2. Critique of Current Highway Design and Operational Criteria

a. Computing Offtracking/Swept Path Width

Several methods are available to determine the path that trucks follow making turns at intersections. The California Truck Offtracking Model (TOM) can be used to plot turning paths and calculate offtracking and swept path widths.[72] A microcomputer program, available from FHWA in IBM PC and Apple computer versions, can plot turning paths but does not display numerical values for offtracking and swept path widths.[73,24] The Tractix Integrator can also be used to plot manually a vehicle's turning path.[75] Traditionally, such exercises were done to create "turning templates" for establishing or checking the geometry of intersection turning lanes/roadways.[76] The Western Highway Institute model can be used to compute fully developed offtracking, a steady state value that may be larger than the actual offtracking value in the early portion of a turn.[77] With the above techniques, various vehicle configurations can be simulated, their paths determined, and the effects of vehicle variations evaluated.

All of the methods for determining offtracking that are discussed above represent only the low-speed component of offtracking on level pavements. In fact, offtracking is also a function of vehicle speed and pavement cross-slope. A new model which quantifies these effects was developed in the present study and is presented in appendix C in volume II. However, the magnitude of the speed and superelevation effects is not usually large enough to be considered in the design of low-speed urban intersections. The role of these factors in determining pavement widening on horizontal curves is discussed in section III-L.

b. Intersection Channelization

Turning characteristics of large trucks, such as offtracking and swept path width, require special consideration in the design of at-grade intersections.[78] If the curb radius is large enough so that trucks can make right turns without encroaching on adjacent lanes, the paved area at the intersection can become so large that through drivers may not understand where to position their vehicle. In such instances, it becomes necessary to construct a channelizing island to properly control traffic. If the curb radius is so small that trucks cannot make right turns without encroaching on adjacent lanes, the truck either encroaches and interferes with adjacent traffic or it does not encroach and its rear wheels run over and possibly damage the curb and/or shoulder. In addition, the truck's front overhang may strike those traffic control devices located near the outside of its turning path, or the trailer's right rear tire may strike those devices located near the inside of its turning path when offtracking. The following discussion addresses these concerns and presents at-grade channelization guidelines based on a recent Texas study of right-turn maneuvers by trucks larger than the design vehicles used in the AASHTO Green Book.[79]

Design Vehicles: The design vehicles that were selected for the Texas study were two singles, two doubles, and one triple. One vehicle, the WB-50, was the same as the design vehicle configuration defined in the AASHTO Green Book, and was used for comparison. The tractor used in each combination was assumed to have a 16-ft (5-m) wheelbase with the cab placed behind the engine. This particular tractor was selected because of its longer wheelbase, typical of cab-behind-engine tractors. The five design vehicles used in the Texas study are presented in table 58; the dimensions chosen for each vehicle were based on data from the literature. The dimensions for the WB-55 and WB-70 trucks differ slightly from the design vehicles recommended in the present study in tables 3 and 4. The offtracking characteristics of the WB-55 design vehicle differ only slightly from the STAA single with 48-ft (14.6-m) trailer in tables 3 and 4, and the offtracking characteristics of the WB-70 differ only slightly from those of the STAA doubles in tables 3 and 4.

Intersection Geometrics: In addition to the design vehicles, other parameters considered in the Texas study were curb return radius and degree of turn. The values for curb return radius evaluated were those specified in table III-19 of the AASHTO Green Book. Sets of the various radii were drawn for turning angles of 60°, 75°, 90°, 105°, 120°, and 180°.

Findings: Computer simulation (TOM) runs were made in the Texas study for each of the different scenarios. The study's results included minimum turning radii, turning templates, cross street width occupied, swept path widths, and channelization guidelines.

Minimum Turning Radii: The minimum turning radii of the outside and inside wheel paths for each of the five design vehicles are given in table 59. The values for the WB-50 vary slightly from those in the AASHTO Green Book due to shorter tractor and longer trailer axle spacings. The minimum turning radii and the transition lengths shown here and in the Green Book are for turns made at less than 10 mi/h (16 km/h). This assumption minimizes the effects of driver characteristics and the slip angles of wheels.

Table 58. Design vehicle dimensions used in Texas study.[79]

		Dimensions (ft)												
		Overall			Overhang									
Design vehicle type	Symbol	Ht.	Width	Length	Front	Rear	WB_1	WB_2	S	T	WB_3	S	T	WB_4
Combination trucks:														
Semitrailer	WB-50	13.5	8.5	55	3	2	16	34.0	-	-	-	-	-	-
Large semitrailer	WB-55	13.5	8.5	60	3	2	16	39.1	-	-	-	-	-	-
Semitrailer-trailer	WB-70	13.5	8.5	75	3	2	16	20.0	2.5	7.5	23.0	-	-	-
Large semitrailer-trailer	WB-105	13.5	8.5	110	3	2	16	37.3	6.7	6.3	37.8	-	-	-
Semitrailer-trailer-trailer	WB-100	13.5	8.5	105	3	2	16	21.9	3.0	6.2	22.3	3.0	6.2	22.3

WB_1, WB_2, WB_3, and WB_4 are effective vehicle wheelbases.
S is the distance from the rear effective axle to the hitch point.
T is the distance from the hitch point to the lead effective axle of the following unit.
Note: 1 ft = 0.305 m.

Table 59. Minimum turning radii of design vehicles in Texas study.[79]

Design vehicle type	Semitrailer combination	Semitrailer combination (large)	Semitrailer-full trailer combination	Semitrailer-full trailer combination (large)	Semitrailer-full trailer-full trailer-combination
Symbol	WB-50	WB-55	WB-70	WB-105	WB-100
Configuration	3-S2	3-S2	3-S1-2	3-S2-4	2-S1-2-2
Minimum turning radius (ft)	45	50	50	65	55
Minimum inside radius (ft)	20.5	19	24.3	25.8	25.6

Note: 1 ft = 0.305 m

Channelization Guidelines: Table 60 is similar to table IX-4 in the AASHTO Green Book and contains minimum designs and channelization guidelines for turning roadways. Channelization should only be used at an intersection if the curb radius and optimum truck turning radius permit the use of an island with an area of at lest 100 ft^2 (9.3 m^2), the minimum size of channelized island recommended by AASHTO. The parameters which govern the design of a channelized intersection are angle of turn, design vehicle, curb radius, width of lane, and approximate island size. For each design vehicle, table 60 lists a suggested island size and width of turning lane at each angle of turn that might need channelization. As the curb return radius increases towards 200 ft (61 m), the area of the island becomes larger and the width of the turning lane decreases. The size of islands for the larger turning angles indicates the size of the otherwise unused and uncontrolled areas of pavement that were eliminated by the use of islands. Turning roadways for flat-angle turns, less than 75°, involve relatively large radii and require designs to fit site controls and traffic conditions.

Because the truck configurations follow a spiral path into a curve, it would be desirable to fit the edge of the pavement closely to the minimum path of the design vehicle by using three-centered compound curves or simple curves with tapers to minimize the amount of unused pavement. The unnecessarily wide turning lane widths in table 60 are an indication that simple radius curves are not well suited to the turning paths of large trucks.

3. Sensitivity Analysis

Current design policy establishes minimum turning paths of a truck based on the boundaries traced by outer front overhang and the sharpest turning radius of the right inner rear wheel. The critical vehicle characteristics in design of intersection channelization and geometrics include overall width, effective wheelbase length, and limits of the turning mechanism.

The Caltrans TOM model was used to develop plots of offtracking data for the design vehicles recommended in this study.[72] Plots were developed for the following design vehicles:

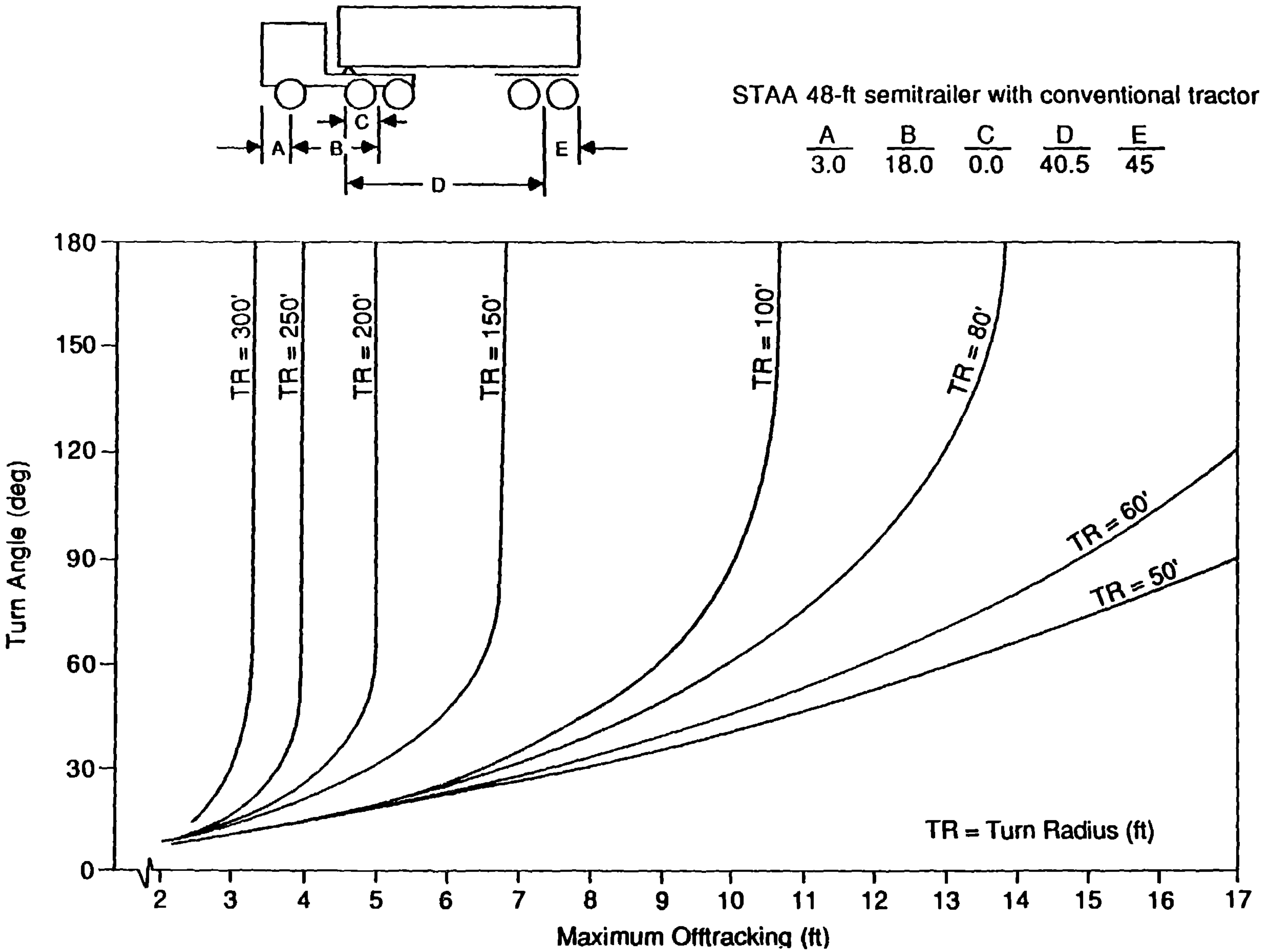
- STAA single with 48-ft (14.6 m) trailer (see figure 35)
- Long single with 53-ft (16.2 m) trailer (see figure 36)
- STAA double with cab-over-engine tractor (see figure 37)
- STAA double with conventional tractor (see figure 38)

Each figure shows a graph of the offtracking for a particular design vehicle as a function of the turn radius and the turn angle. The swept path width can be obtained by adding 7.58 ft (2.31 m) to the indicated value of offtracking. The channelization and turning lane width requirements for the STAA single with 48-ft (14.6-m) trailer are represented approximately by the WB-55 truck in table 60; the requirements for the STAA doubles are approximated by the WB-70 truck in table 60. In both of these cases, the swept path width for the design vehicle in table 60 is approximately 2 ft (0.6 m) greater than the swept path width for the corresponding design vehicle used in this study.

Table 60. Minimum designs and channelization guidelines for turning roadways.[79]

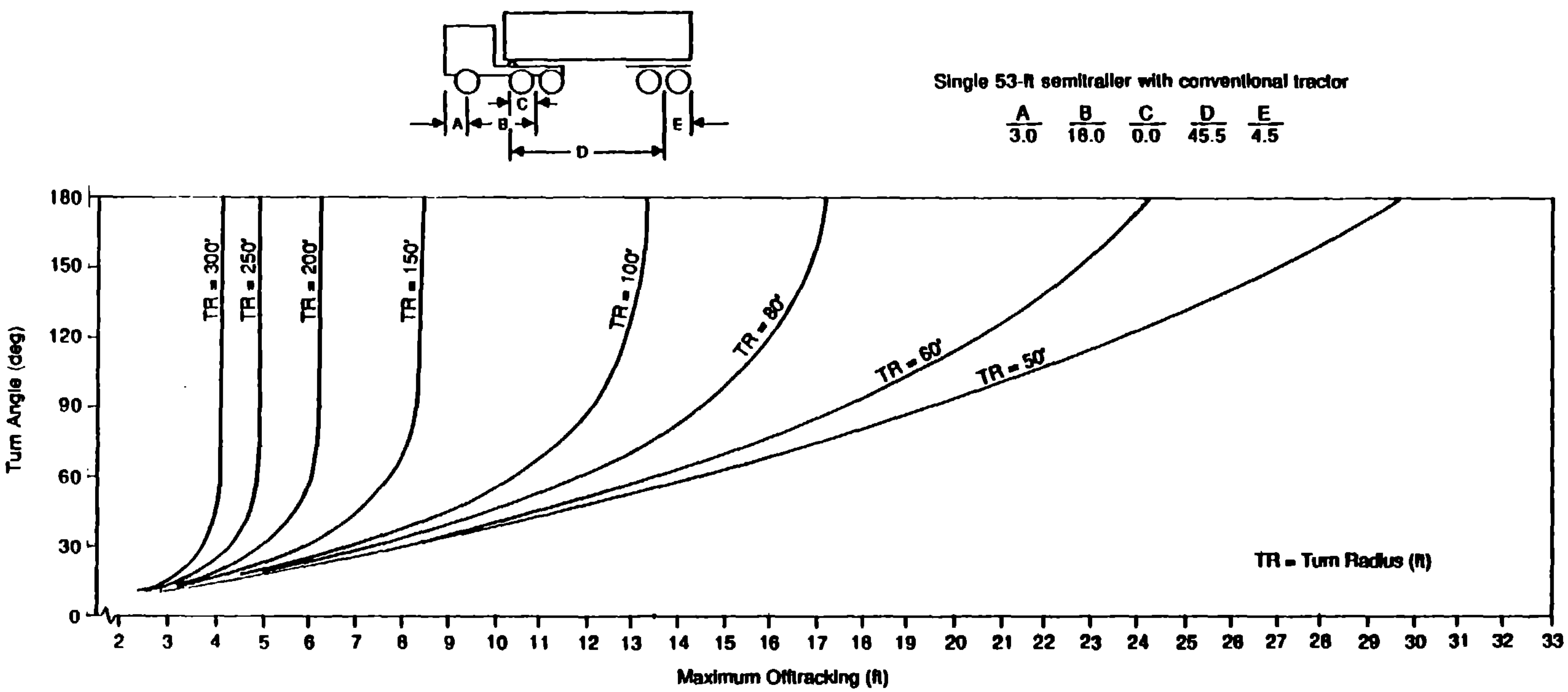
Angle of turn (degrees)	Design vehicle	Curb radius (ft)	Width of turning lane (ft)	Approximate island size (ft^2)
60	WB-50	200	27	250
	WB-55	200	22	160
	WB-70	200	22	160
	WB-100	200	27	160
	WB-205	-	-	-
75	WB-50	150	28	320
	WB-55	150	30	160
	WB-70	150	23	200
	WB-100	200	34	300
	WB-105	-	-	-
90	WB-50	150	30	670
	WB-55	200	38	900
	WB-70	150	22	560
	WB-100	200	40	900
	WB-105	200	54	260
105	WB-50	150	32	980
	WB-55	150	41	740
	WB-70	150	31	1,320
	WB-100	200	41	1,940
	WB-105	200	57	940
120	WB-50	150	40	1,640
	WB-55	200	45	3,400
	WB-70	150	39	1,600
	WB-100	200	48	2,580
	WB-105	200	60	1,740

Note: 1 ft = 0.305 m



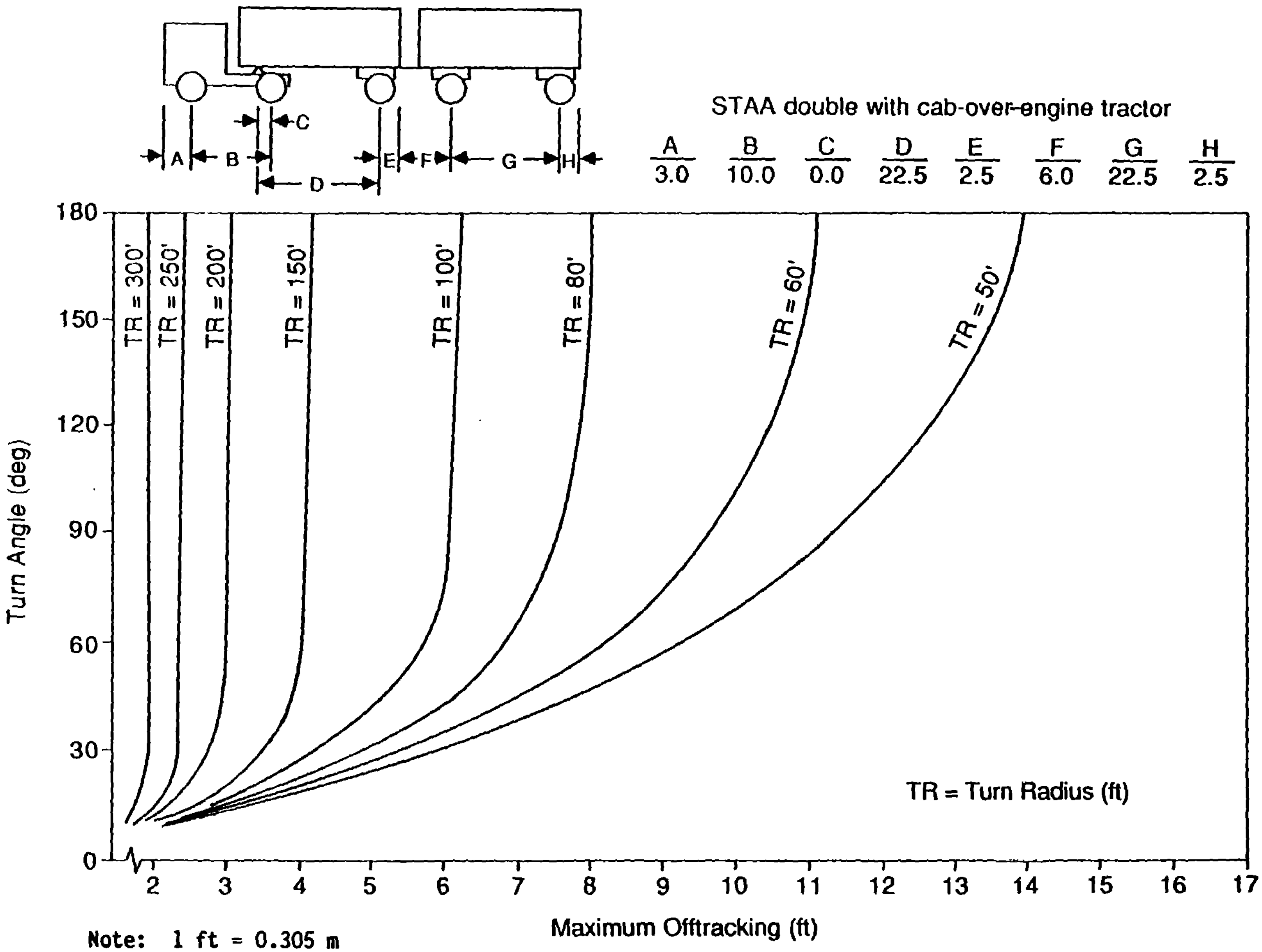
Note: 1 ft = 0.305 m

Figure 35. Offtracking plot for STAA single 48-ft (14.6-m) semitrailer truck with conventional tractor.



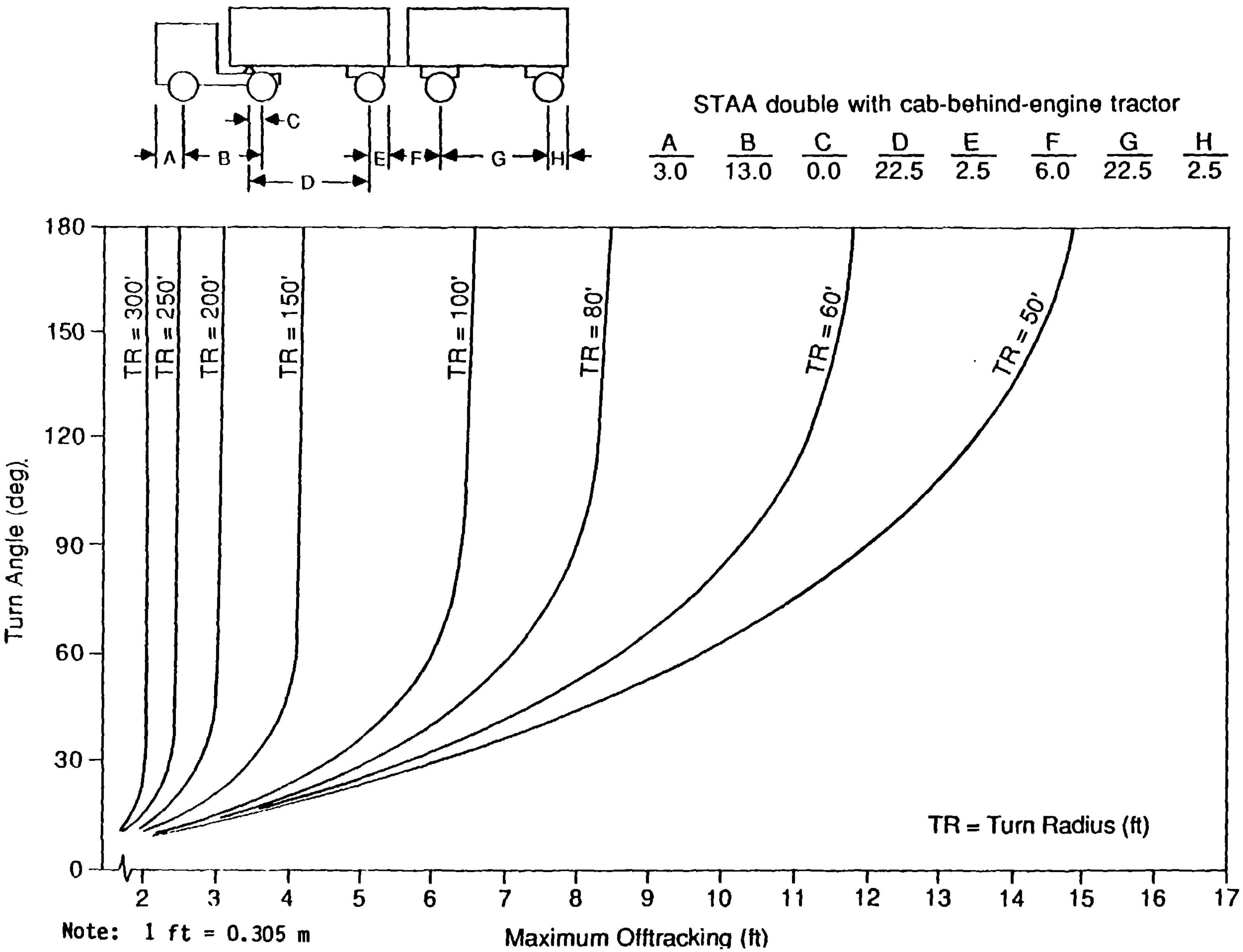
Note: 1 ft = 0.305 m

Figure 36. Offtracking plot for long single 53-ft (16.2-m) semitrailer truck with conventional tractor.



Note: 1 ft = 0.305 m

Figure 37. Offtracking plot for STAA double-trailer truck with cab-over-engine tractor.



Note: 1 ft = 0.305 m

Figure 38. Offtracking plot for STAA double-trailer truck with cab-behind-engine tractor.

Thus, table 60 provides estimates of turning lane width and island size that are slightly conservative.

The dimensions of the design vehicles used to develop the offtracking plots in figures 35 through 38 are given in table 4. That table shows dimension D for the 53-ft (16.2-m) semitrailer truck as 45.5 ft (13.8 m). This dimension is appropriate for a 53-ft (16.2-m) truck with the rear axles positioned as close to the rear of the truck as possible. Thus, figure 36 is appropriate for a truck in this configuration. It should be recognized, however, that the rear axles of many trucks can be moved forward to reduce offtracking. Some States have restricted the kingpin-to-rear-axle distance for 53-ft (16.2-m) trailers to a maximum of 41 ft (12.5 m), which provides nearly the same offtracking as the 48-ft (14.6-m) truck whose offtracking performance illustrated in figure 35.[6] Restriction of the kingpin-to-rear-axle distance is not enough, by itself, to provide for safe truck operations unless the front and rear overhang of the trailer are also restricted. Long front or rear overhang can lead to swingout by the left rear or left front corner of the trailer in a right-hand turn and underride problems if the trailer is struck in the rear by a passenger car.

4. Recommended Revisions to Highway Design and Operational Criteria

The recommended STAA design vehicles in tables 3 and 4 have greater offtracking and will require greater swept path widths than the design vehicles used in the current AASHTO criteria. There is a clear need to revise these criteria to include larger design vehicles in the AASHTO Green Book and to include updated turning templates and swept path width guidelines identified in section II-A of this report.

Table 61 documents the additional pavement construction costs of accommodating larger trucks at urban intersections. The table shows the additional paved area and the additional pavement cost to accommodate 45-, 48-, and 53-ft (13.7-, 14.6-, and 16.2-m) tractor-semitrailer trucks in comparison to an intersection designed for the WB-50 design vehicle. The pavement areas and costs in the table are for one quadrant of a 90° intersection and should be multiplied by 4 to represent all quadrants of a conventional four-leg intersection. Of course, higher costs would be incurred at some locations if additional right of way were required or if sidewalks, signals, or utility poles needed to be moved in a rehabilitation project at an existing site.

5. Summary

Intersection and channelization geometrics should be based on the low-speed offtracking characteristics of the larger design vehicles identified above. The offtracking characteristics of these vehicles are documented above and in appendix C in volume II.

Table 61. Additional pavement construction costs to accommodate design vehicles larger than in AASHTO WB-50 truck at urban intersections.

Turning radius (ft)	Additional paved area per quadrant (ft^2)			Additional construction cost[a] per quadrant		
	45-ft semitrailer	48-ft semitrailer	53-ft semitrailer	45-ft semitrailer	48-ft semitrailer	53-ft semitrailer
50	900.8	1,225.1	1,849.6	$ 2,620	$ 3,570	$ 5,380
60	1,095.6	1,423.0	2,283.0	3,190	4,140	6,640
80	1,243.4	1,673.0	2,939.0	3,620	4,870	8,550
100	1,498.1	2,085.6	3,319.3	4,360	6,070	9,660
150	1,601.8	2,242.5	3,752.8	4,660	6,530	10,920
200	1,631.6	2,249.6	3,732.8	4,750	6,550	10,860
250	1,554.3	2,331.5	3,730.3	4,520	6,790	10,860
300	1,403.1	2,245.0	3,648.1	4,080	6,533	10,620

[a] Based on cost of $2.91/ft^2 ($31.28/m^2) for flexible pavement.
Note: 1 ft = 0.305 m.

F. Railroad-Highway Grade Crossing Sight Distance

1. Current Highway Design and Operational Criteria

The FHWA Railroad-Highway Grade Crossing Handbook and the AASHTO Green Book use the same principles to determine sight distance requirements at railroad-highway grade crossings.[1,80] Both consider sight distance requirements for a moving highway vehicle and for a highway vehicle accelerating from a stop at the crossing, as shown in figure 39. For the moving vehicle situation, the sight distance, d_H, along the highway must, as a minimum, equal the stopping sight distance for the design speed of the approach. The sight distances along the track for this situation are the distances traveled by the train during the time the highway vehicle traverses both the highway distance, d_H, and the distance to clear the crossing. For the stopped vehicle situation, the highway vehicle starts from a minimum safe distance from the crossing. The distances along the track for this situation are those traveled by the train at various speeds while the highway vehicle accelerates and just clears the crossing. Each of these cases is addressed below.

a. Sight Distance Along the Highway for a Moving Vehicle

The minimum sight distance along the highway, d_H, is measured from the nearest rail to the driver of a vehicle. It is the sum of the minimum stopping sight distance and the minimum clearance distance between the tracks and the driver after the vehicle stops. This sight distance allows an approaching vehicle to avoid collision by stopping without encroaching on the crossing area. The minimum safe sight distance formula used in the FHWA Handbook and the AASHTO Green Book is:

$$d_H = 1.47\ V_v t_{pr} + \frac{V_v^2}{30f} + D + d_e \qquad (58)$$

where: d_H = sight distance along the highway (ft)

V_v = speed of vehicle (mi/h)

t_{pr} = perception/reaction time of driver (assumed: t_{pr} = 2.5 s)

f = coefficient of friction used in braking (equal to those used for stopping sight distance criteria in table 20)

D = clearance distance from front of vehicle to the nearest rail (assumed: D = 15 ft or 5 m)

d_e = distance from driver's eye to the front of vehicle (ft) (assumed: d_e = 10 ft or 3 m)

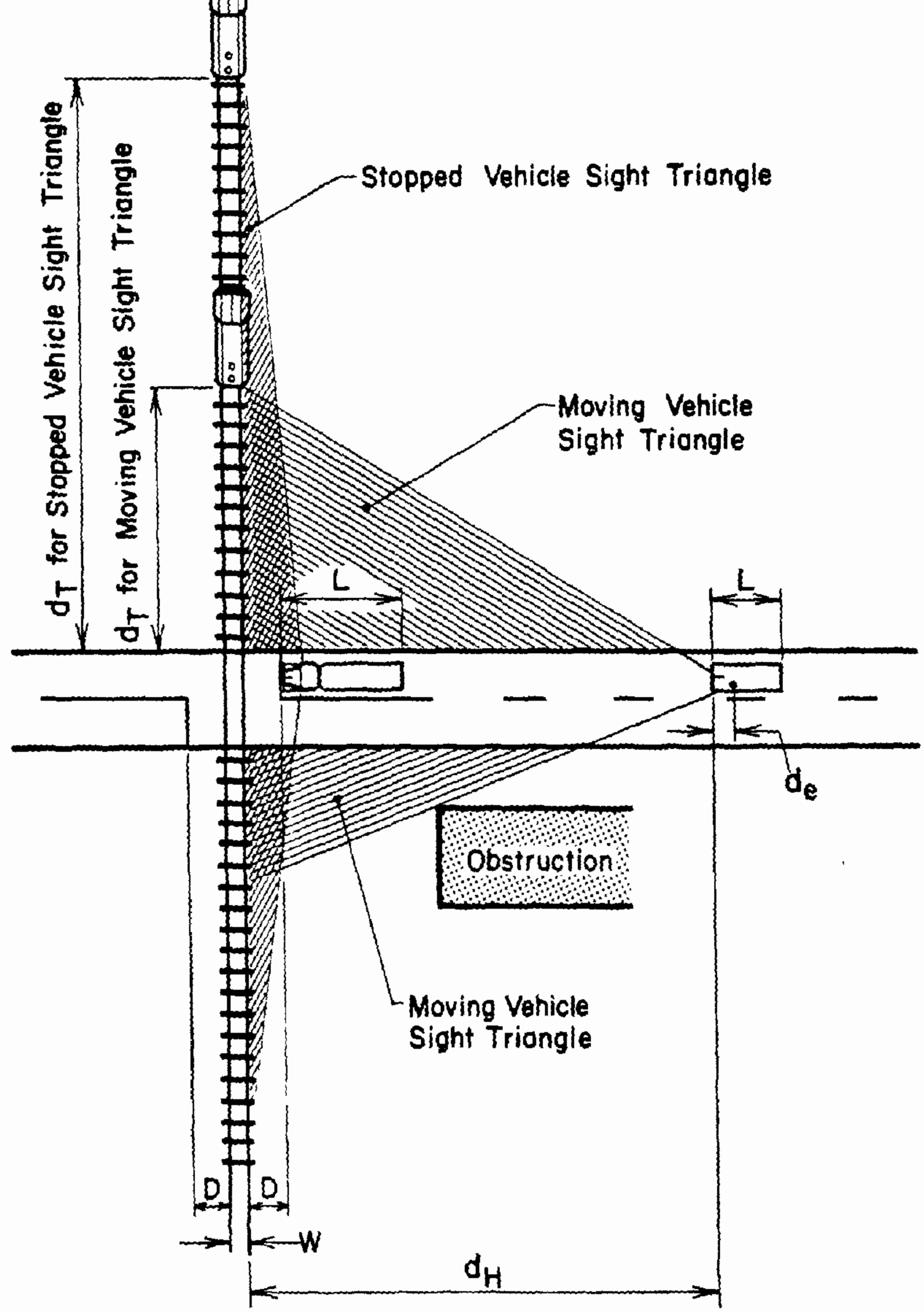


Figure 39. Dimensions considered in railroad-highway grade crossing sight distance.[80]

b. Sight Distance To and Along Tracks for a Moving Vehicle

The legs of the clear sight "triangle" are formed by the distance of the vehicle from the track, d_H, and the distance of the train from the crossing, d_T. The equation for d_H is presented above. The minimum distance along the track, d_T, is measured from the nearest edge of the highway travel lane being considered to the front of the train. It is the product of the train speed and the time required by the highway vehicle to traverse the highway leg (d_H) and then to clear the crossing. The distance d_T is computed in the FHWA Handbook and the AASHTO Green Book as:

$$d_T = \frac{V_t}{V_v}\left[1.47\ V_v t_{pr} + \frac{V_v^2}{30f} + 2D + L + W\right] \qquad (59)$$

where: d_T = sight distance along the railroad tracks for a moving vehicle (ft)

V_t = speed of train (mi/h)

V_v = speed of vehicle (mi/h)

t_{pr} = perception-reaction time of driver (s) (assumed: $t = 2.5$ s)

f = coefficient of friction used in braking (see table 20)

D = clearance distance from front of vehicle to the nearest rail (assumed: $D = 15$ ft or 5 m)

L = length of vehicle (ft) (assumed: $L = 65$ ft or 20 m)

W = distance between outer rails (assumed for a single track: $W = 5$ ft or 1.5 m)

It is assumed that the truck is crossing a single track at 90° in level terrain. Users are cautioned that adjustments should be made for unusual vehicle lengths and acceleration capabilities, multiple tracks, skewed crossings, and grades.

c. Sight Distance Along Tracks for a Stopped Vehicle

The third case provides sight distance needed to allow a stopped vehicle to accelerate and cross the tracks before the train reaches the crossing. It considers the perception-reaction time of the driver and vehicle characteristics such as the maximum speed of the vehicle in its initial gear, the acceleration capability of the vehicle, and the length of the vehicle. The required sight distance, d_T, along the tracks is determined in the FHWA Handbook and the AASHTO Green Book as:

$$d_T = 1.47\ V_t \frac{V_g}{a_1} + \frac{L + 2D + W - d_a}{V_g} + J \qquad (60)$$

where: d_T = sight distance along the railroad tracks for a stopped vehicle (ft)

V_t = speed of train (mi/h)

V_g = maximum speed of vehicle in first gear (ft/s) (assumed: V_g = 8.8 ft/s or 2.7 m/s)

a_1 = acceleration rate of vehicle in first gear (assumed: a_1 = 1.47 ft/s^2 or 0.45 m/s^2)

L = length of vehicle (ft) (assumed: L = 65 ft or 20 m)

W = distance between outer rails (assumed: W = 5 ft or 1.5 m)

D = clearance distance from front of vehicle to the nearest rail (ft) (assumed: D = 15 ft or 5 m)

J = sum of perception-reaction time of driver and time required to actuate the clutch or an automatic shift (assumed: J = 2.0 s)

d_a = distance vehicle travels while accelerating to maximum speed in first gear (ft) = $V_g^2/2a_1$

The assumptions of a single track, a 90° crossing, and level terrain are made and the same cautions apply as for the previous case.

2. Critique of Current Highway Design and Operational Criteria

A review of driver characteristics in a 1984 FHWA study addressed changes in sight distance requirements with changes in the driver characteristics.[81] The driver characteristic reviewed for railroad-highway grade crossing sight requirements was perception-reaction time. Their findings indicate that the sight distance requirements are relatively insensitive to changes in the perception-reaction time.

A review of the cases in the AASHTO Green Book in the 1984 FHWA study found the formulation for calculating minimum corner sight triangle for a moving vehicle to be correct and reasonable.[33] They also concluded that the concept for determining the minimum sight distance along a track for a stopped vehicle was correct and adequately addressed both the driver and vehicle requirements.

A 1976 Canadian study reported that the lack of uniformity in driver behavior indicates a high level of decisional uncertainty with respect to the correct response to grade crossings and may be a major cause of crossing accidents.[82] Vehicle speed variations were higher as the distance to the crossing decreased. Specific speed variations for trucks were not reported.

NCHRP Report 50 identified factors influencing safety at railroad-highway grade crossings based on a sample of 3,627 accidents.[83] One third of the accidents involved trains, one third occurred when a train was present but was not involved, and one third occurred when no train was present. They reached the following conclusions:

- Comparison of the distribution of vehicle speeds at the crossing and prior to the influence of the crossing indicated a definite reduction in average speed and fewer vehicles within the 10-mi/h (16-km/h) pace. These conditions were believed to contribute significantly to multiple-vehicle accidents at crossings.

- While trucks account for approximately 11 percent of the vehicles involved in all types of motor vehicle accidents, they are involved in 20.4 percent of the train-involved crossing accidents.

- High truck involvement in accidents may be attributable to their length and the fact that they occupy the crossing longer than passenger cars.

The sight distance criteria for grade crossings are based on the same friction coefficients used for AASHTO stopping sight distance criteria. The friction coefficients represent the deceleration rates used by passenger cars in locked-wheel braking on a wet pavement. Trucks cannot safely make locked-wheel stops without risking loss of control. Section II-A of this report documents that the deceleration rates used by trucks in making controlled stops are generally lower than the deceleration rates used by passenger cars making locked-wheel stops.

The FHWA Handbook does not cite any documentation as the basis for its assumptions concerning the maximum speed of the vehicle in first gear and the acceleration rate of the vehicle. However, the assumed values appear reasonable.

3. Sensitivity Analysis

The current sight distance policies directly or indirectly use different vehicle types as the design vehicle. The sight distance along the highway for a moving vehicle is derived assuming a passenger car as the design vehicle, since the deceleration rates are based on locked-wheel braking by a passenger car, on a wet pavement. The derivation for sight distances along the tracks for a moving vehicle mixes the characteristics of different vehicle types by using passenger car deceleration rates and a 65-ft (20-m) vehicle length (typical of a WB-60 truck). The design vehicle for sight distance along tracks for a stopped vehicle is a 65-ft (20-m) truck with reasonable assumptions for both acceleration and the maximum speed in first gear.

The following sensitivity analysis determines the railroad-highway grade crossing sight distance requirements using consistent data for trucks. This sensitivity analysis is a simple extension of the existing sight distance models to reflect current truck characteristics and performance. Table 62 summarizes the models and the parameters currently used in them. They include a driver-related characteristic (perception-reaction time) and several vehicle-related characteristics (stopping sight distance, vehicle length, and maximum speed and acceleration in first gear). Table 63 presents the range of values of the vehicle-related parameters (including vehicle length, stopping sight distance, and vehicle acceleration) that have been varied in the sensitivity analysis.

Truck lengths of 70 and 75 ft (21 and 23 m) were used in the analyses. The stopping sight distances used are those presented in table 23 for both the worst and best performance drivers (62 to 100 percent driver control efficiency). The Gillespie model for clearance times for trucks crossing an intersection is used to determine the times for stopped vehicles to clear the crossing.[25]

a. Sight Distance Along Highway for a Moving Vehicle

The sight distance along the highway ahead to the crossing (d_H) increases significantly in comparison to the current FHWA and AASHTO criteria when the increased stopping sight distances of trucks are considered. Table 64 presents the required sight distances for current criteria in comparison to trucks with the worst and best performance drivers. The results shown in table 64 are illustrated in figure 40. While the difference is minimal for a truck with the best performance driver (between 7 to 22 percent increase in sight distance), substantial increases in sight distances (between 30 and 54 percent) are required for a truck with the worst performance driver.

b. Sight Distance Along Tracks for a Moving Vehicle

The sensitivity analysis of the sight distance requirements along the track from the crossing (d_T) for a 70-ft (21-m) truck found similar results (see tables 65 and figure 41). This truck requires 23 percent more sight distance at 20 mi/h (32 km/h) and up to 47 percent more at 70 mi/h (113 km/h) for the worst performance driver. The best performance driver in a 70-ft (21-m) truck requires at most a 20 percent increase in sight distance. A 75-ft (23-m) truck requires similar increases in sight distance (a maximum 22 percent increase for the best performance driver and a 49 percent increase for the worst performance driver). Not only does the greater truck length increase the required sight distance, but the braking distance for the worst performance driver for both truck lengths also significantly increased the required sight distance.

Table 62. Summary of parameters for railroad-highway grade crossing sight distance

Equations	Perception-reaction time (s)	Stopping sight distance (SSD): Speed (mi/h)	Stopping sight distance (SSD): AASHTO[1] SSD (ft)	Parameters used to determine sight distance[a]: d_e (ft)	D (ft)	W (ft)	L (ft)	V_g (ft/s)	a_1 (ft/s)
Sight distance along a highway	$t_{pr} = 2.5$	20	125	10	15	NA	NA	NA	NA
		30	200						
$d_H = SSD + D + d_e$		40	325						
		50	475						
$d_H = SSD + 15 + 10$		60	650						
		70	850						
$d_H = SSD + 25$									
Sight distance along tracks for a moving vehicle	$t_{pr} = 2.5$	20	125	NA	15	5	65 (WB-60)	NA	NA
		30	200						
		40	325						
		50	475						
$d_T = \frac{V_t}{V_v} [SSD + 2D + L + W]$		60	650						
		70	850						
$d_T = \frac{V_t}{V_v} [SSD + 2 * 15 + 65 + 5]$									
$d_T = \frac{V_t}{V_v} [SSD + 100]$									

Table 62. Summary of parameters for railroad-highway grade crossing sight distance. (continued)

Equations	Perception-reaction-time (s)	Stopping sight distance (SSD)	Parameters used to determine sight distance[a]					
			d_e (ft)	D (ft)	W (ft)	L (ft)	V_g (ft/s)	a_1 (ft/s)
Sight distance along tracks for a stopped vehicle	J = 2.0	NA	NA	15	5	65 (WB-60)	8.8	1.47

$$d_T = 1.47V_t[\frac{V_G}{a_1} + \frac{L + 2D + W - d_a}{V_g} + J] \qquad d_a = \frac{V_g^2}{2a_1}$$

$$d_T = 1.45V_t\ [\frac{8.8}{1.47} + \frac{65 + 2 * 15 + 5 - (8.8^2/(2 * 1.47))}{8.8} + 2.01]$$

$$d_T = 16.9V_t$$

a d_e = distance from driver's eye to front of vehicle (ft)

D = clearance distance from front of vehicle to nearest rail (ft)
W = distance between outer rails (ft)
L = length of vehicle (ft)
V_g = maximum speed of vehicle in first gear (ft/s)
a_1 = acceleration rate of vehicle in first gear (ft/s^2)

Note: 1 ft = 0.305 m
1 mi = 1.61 km

Table 63. Summary of parameters varied in sensitivity analysis for railroad-highway grade grossing sight distance

Consideration	Vehicle length (ft)	Stopping sight distance (SSD)			Assumptions
			Truck driver performance		
		Speed (mi/h)	Worst	Best	
Sight distance along a highway	NA				NA
$d_H = SSD + D + d_e$		20	150	125	
		30	300	250	
$d_H = SSD + 15 + 10$		40	500	375	
		50	725	525	
$d_H = SSD + 25$		60	975	700	
		70	1,275	900	
Sight distance to and along tracks for a moving vehicle	70-ft tractor semi-trailer truck 75-ft tractor semi-trailer-full trailer truck (double bottom)		Truck driver performance		NA
		Speed (mi/h)	Worst	Best	
		20	150	125	
		30	300	250	
		40	500	375	
		50	725	525	
		60	975	700	
		70	1,275	900	

$$d_T = \frac{V_t}{V_v} (SSD + 2D + L + W)$$

$$d_T = \frac{V_t}{V_v} (SSD + 2 * 15 + L + 5)$$

$$d_T = \frac{V_t}{V_v} (SSD + 35 + L)$$

Table 63. Summary of parameters varied in sensitivity analysis for railroad-highway grade crossing sight distance. (continued)

Consideration	Vehicle length (ft)	Stopping sight distance (SSD)	Additional assumptions
Sight distance along tracks for a stopped vehicle	70-ft tractor semitrailer truck 75-ft tractor semitrailer-full trailer truck (double bottom)	NA	t_c = time to clear hazard zone (from the Gillespie model[25])

$$d_T = 1.47\ V_t\ (t_c + J)$$

$$D_T = 1.47\ V_t\ [0.682 * (2 * D + W + L)/Vmg + 3.0 + 2.0]$$

$$d_T = 1.47\ V_t\ [0.682 * (2 * 15 + 5 + L)/8 + 3.0 + 2.0]$$

$$d_T = V_t\ [0.125\ L + 11.73]$$

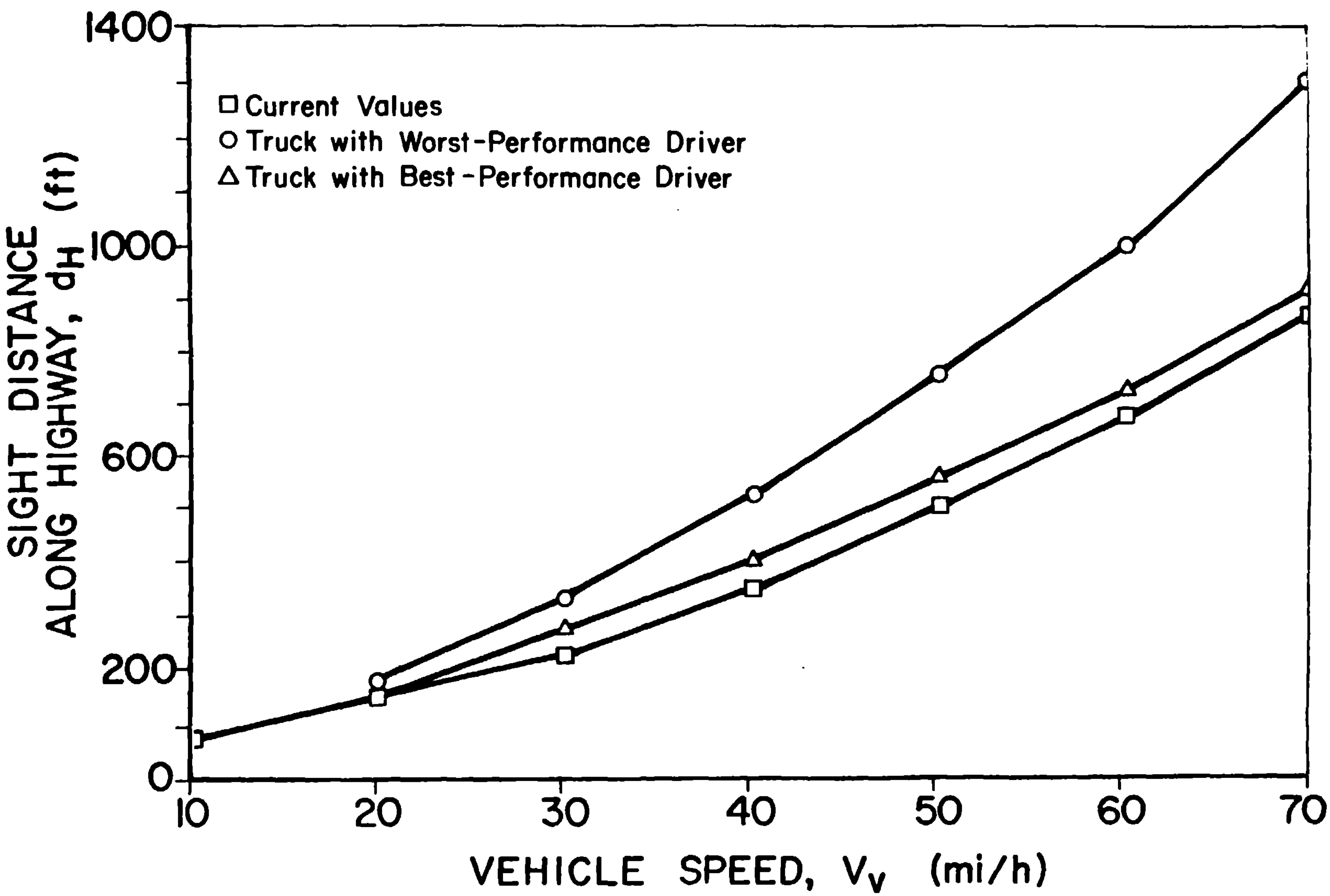
Note: 1 ft = 0.305 m
1 mi = 1.61 km

Table 64. Sensitivity analysis for sight distance along a highway (d_H) at railroad-highway grade crossing.

	Vehicle speed, V_v (mi/h)					
	20	30	40	50	60	70
	d_H (ft)	d_H (ft)	d_H (ft)	d_H (ft)	d_H (ft)	d_H (ft)
Current values	135	225	340	490	660	865
Sight distance for a truck with worst-performance driver	175	325	525	750	1,000	1,300
Sight distance for a truck with best-performance driver	150	275	400	550	725	925

All calculated sight distances are rounded up to the next higher 5-ft (1.5-m) increment.

Note: 1 mi = 1.61 km
1 ft = 0.305 m



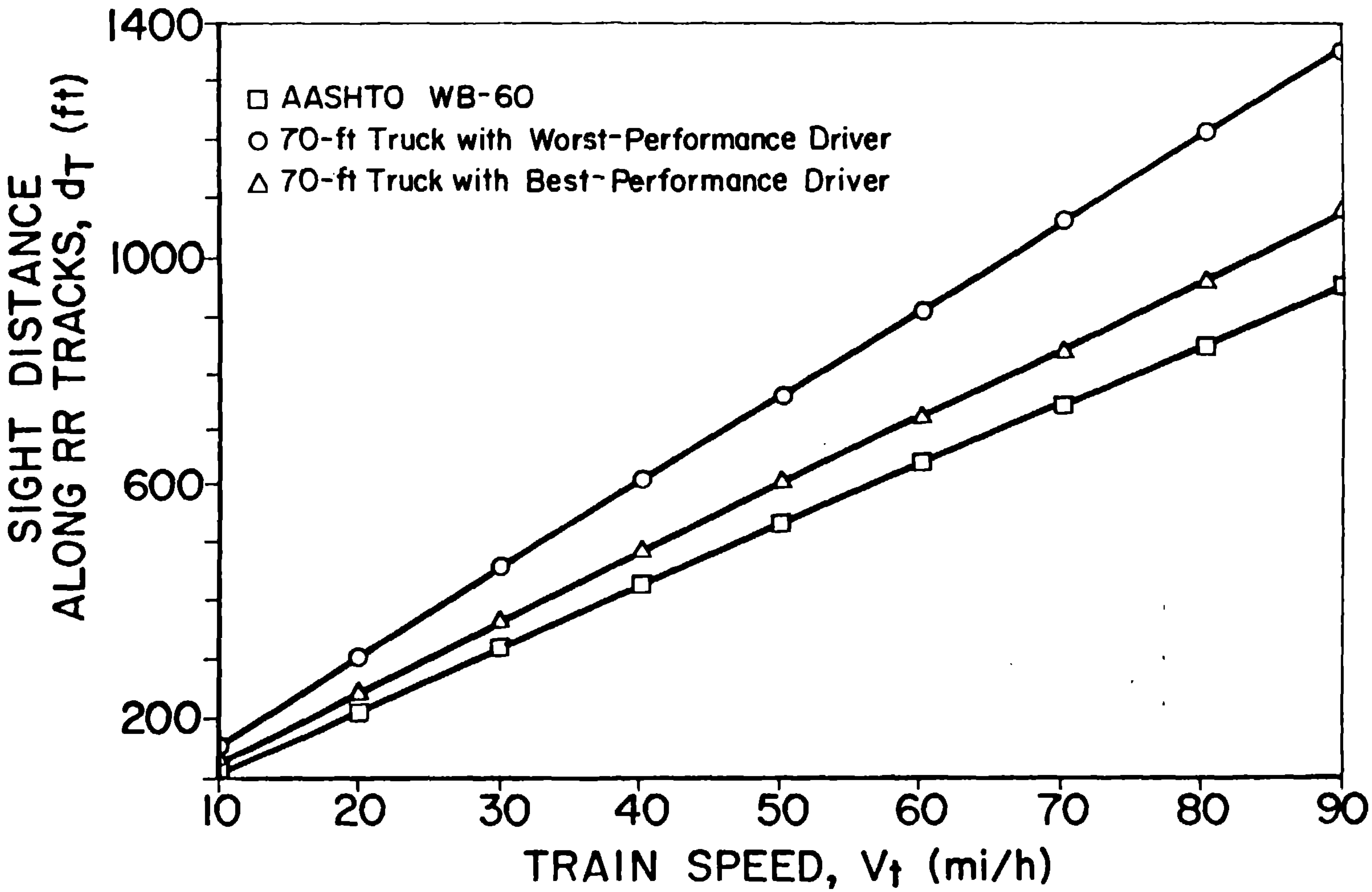
Note: 1 ft = 0.305 m
1 mi = 1.61km

Figure 40. Sensitivity of sight distance along the highway to vehicle speeds.

Table 65. Sensitivity analysis for sight distance to and along tracks (d_T) for a moving vehicle at railroad-highway grade crossings.

Train speed V_t (mi/h)	Vehicle speed, V_v (mi/h) 20 d_T (ft)	30 d_T (ft)	40 d_T (ft)	50 d_T (ft)	60 d_T (ft)	70 d_T (ft)
Current AASHTO procedures using a 65-ft truck						
10	105	100	105	115	125	135
20	210	200	210	225	245	270
30	310	300	310	340	370	405
40	415	395	415	450	490	540
50	520	495	520	565	615	675
60	620	595	620	675	735	810
70	725	690	725	790	860	940
80	830	790	830	900	980	1,075
90	930	930	930	1,010	1,105	1,210
Sight distance for a 70-ft truck with worst-performance driver						
10	128	135	151	166	180	197
20	255	270	303	332	360	394
30	383	405	454	498	540	591
40	510	540	605	664	720	789
50	638	675	756	830	900	986
60	765	810	908	996	1,080	1,183
70	893	945	1,059	1,162	1,260	1,380
80	1,020	1,080	1,210	1,328	1,440	1,577
90	1,148	1,215	1,361	1,491	1,620	1,774
Sight distance for a 70-ft truck with best-performance driver						
10	115	118	120	126	134	144
20	230	237	240	252	268	287
30	345	355	360	378	403	431
40	460	473	480	504	537	574
50	575	592	600	630	671	718
60	690	710	720	756	805	861
70	805	828	840	882	939	1,005
80	920	947	960	1,008	1,073	1,149
90	1,035	1,065	1,080	1,134	1,208	1,292

Note: 1 mi = 1.61 km
1 ft = 0.305 m



Note: 1 ft = 0.305 m
1 mi = 1.61 km

Figure 41. Sensitivity of sight distance along the railroad tracks for a moving vehicle to train speed for a vehicle speed of 40 mi/h (64 km/h)

c. Sight Distance Along Tracks for a Stopped Vehicle

The sight distance requirement along the tracks for a stopped vehicle is not very sensitive to vehicle length. Table 66 and figure 42 present the results of increasing the current design vehicle length of 65 ft (20 m) to 70 and 75 ft (21 and 23 m) and using the by Gillespie model for the time to clear a hazard zone.[25] The sight distance values calculated using AASHTO assumptions of a 65-ft (20-m) truck, 8.8 ft/s (2.7 m/s) for maximum speed of vehicle in first gear, and 1.47 ft/s² (0.45 m/s²) for acceleration rate of the vehicle in first gear are longer than those calculated using a 70- or 75-ft (21- or 23-m) truck length and the Gillespie model for clearance times. The reason for this result is that the Gillespie model provides lower values of clearance time than the current AASHTO model.

4. Recommended Revisions to Design and Operational Criteria

Based on the sensitivity analysis results, a change in the requirements for sight distance along the highway for a moving vehicle is recommended to correspond to any change made in general stopping sight distance requirements to accommodate trucks. As in the case of stopping sight distance requirements, this change is applicable only to higher volume roadways and only if antilock brakes for trucks do not come into nearly universal use. The recommended revisions for stopping sight distance were found to be cost effective only on two-lane highways with truck volumes over 800 trucks/day and freeways with volumes over 4,000 trucks/day. These volume requirements might be relaxed somewhat due to higher truck accident severities at railroad-highway grade crossings.

The existing criteria for sight distance along the tracks for stopped vehicles are adequate to accommodate trucks and do not need to be modified.

5. Summary

The sensitivity analyses demonstrate that increased sight distance for moving vehicles along the highway (d_H) ahead to the railroad-highway grade crossings is needed to accommodate trucks primarily because of their longer braking distances. These recommended revisions are equivalent to those recommended for stopping sight distance in section III-A. These revisions are applicable only to higher volume roadways and only if antilock brake systems for trucks are not required by government regulations and do not come into nearly universal use.

Similar conclusions were reached for sight distance needed along the tracks from the crossing (d_T) for a moving vehicle. Substantially longer sight distances are required for a truck with the worst-performance driver (up to 49 percent increase in sight distance).

In contrast, the current requirements for sight distance required along the tracks for a stopped vehicle were found to be adequate to accommodate trucks.

Table 66. Sensitivity analysis for sight distance along track for a stopped vehicle at railroad-highway grade crossings.

Train speed V_T (mi/h)	Sight distance (d_T) (ft): AASHTO Procedure (WB-60 truck)	70-ft tractor-semi-trailer truck	75-ft tractor-semi-trailer-full trailer truck
10	240	206	212
20	481	412	423
30	721	617	635
40	962	823	847
50	1,202	1,029	1,058
60	1,443	1,235	1,270
70	1,682	1,441	1,482
80	1,924	1,646	1,693
90	2,164	1,852	1,905

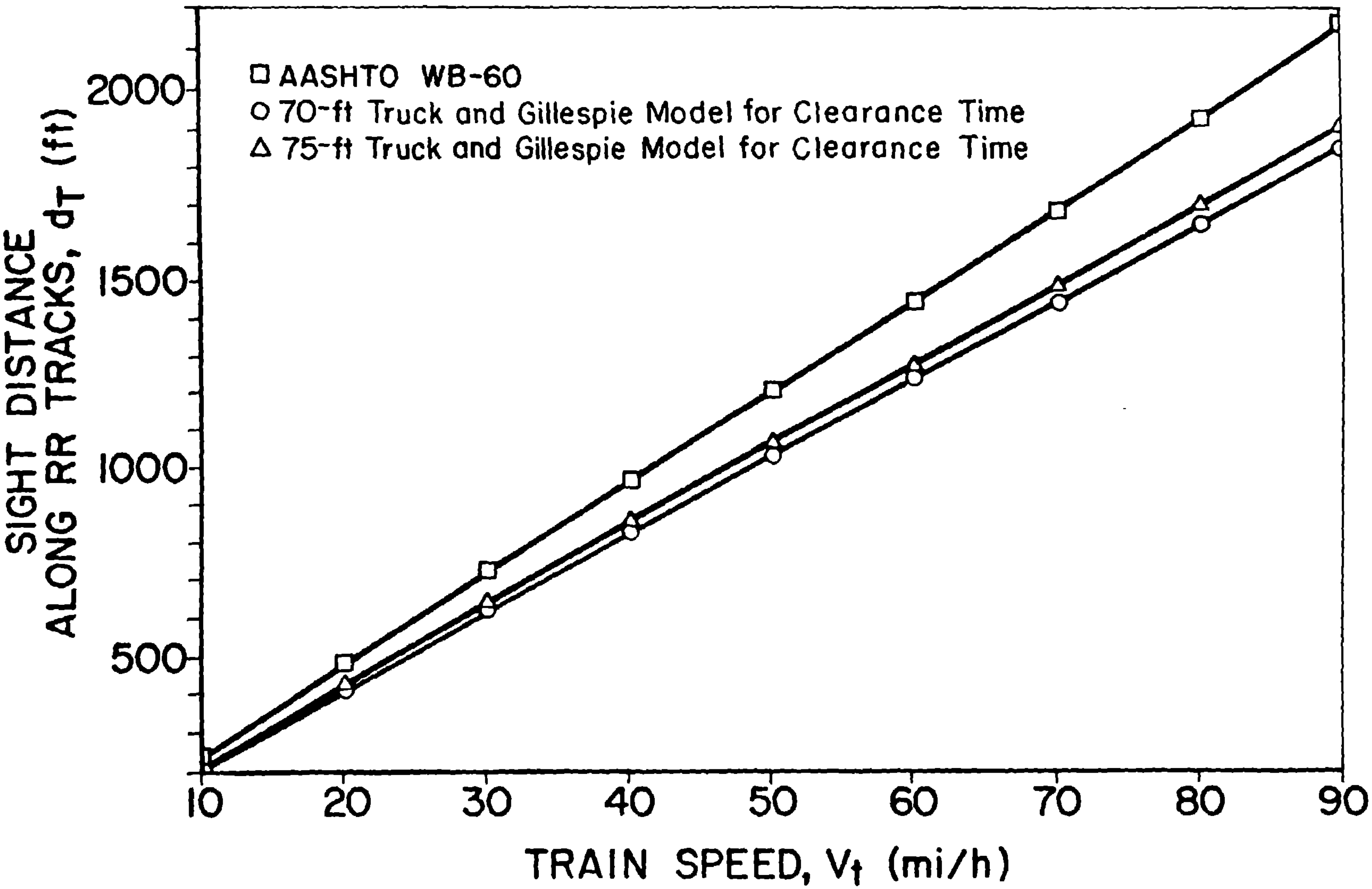
Assumed:

t_c determined from Gillespie model[25]
t_c = 12.0 s for 70-ft (21-m) truck
t_c = 12.4 s for 75-ft (23-m) truck

L_{hz} = 2D + W = 2 * 15 + 5 = 35 ft (10.7 m)

V_{mg} = 8.0 mi/h (12.9 km/h)

Note: 1 ft = 0.305 m
1 mi = 1.61 km



Note: 1 ft = 0.305 m
1 mi = 1.61 km

Figure 42. Sensitivity of sight distance along the railroad tracks to train speed for a stopped vehicle.

G. Crest Vertical Curve Length

1. Current Design and Operational Criteria

Crest vertical curves are designed on the basis of criteria discussed earlier in this report. The primary control in selecting the length of a crest vertical curve is the need to provide stopping sight distance at all points along the curve. For any set of stopping sight distance criteria given in section III-A of this report, such as the AASHTO criteria in table 16 or the truck criteria in tables 23 and 24, the minimum length of a crest vertical curve can be determined from equations (27) and (28).

Drainage needs are also considered in the selection of a crest vertical curve length. The likelihood of drainage problems on a vertical curve can be determined from the rate of vertical curvature, defined as:

$$K = \frac{L}{A} \tag{61}$$

where, K = rate of vertical curve (length per percent change in A)

L = Length of vertical curve (ft)

A = algebraic difference in percent grade

Vertical curves with values of K larger than 167 may experience drainage problems in the level portion of the curve near the crest. The concern about drainage problems does not exclude use of vertical curve values of K larger than 167, but merely requires that drainage be carefully considered on such curves.

Each of the sight distance issues addressed earlier in this report may also affect the design of crest vertical curves. Equations (27) and (28) are also applicable to the provision of passing sight distance if a passing zone is to be maintained over a crest. Decision sight distance, intersection sight distance, or railroad-highway grade crossing sight distance can also be controlling factors in determining crest vertical curve length.

2. Sensitivity Analyses

No sensitivity analyses of truck requirements for crest vertical curve length are presented here because these analyses have already been presented in the analysis of the individual sight distance issues. The minimum crest vertical curve requirements for trucks are presented in table 25 for stopping sight distance, in table 35 for passing sight distance, and in table 41 for decision sight distance.

3. Summary

The design of crest vertical curves is closely related to the sight distance issues addressed earlier in this report. In particular, stopping sight distance is addressed in section III-A, passing sight distance in section III-B, and decision sight distance in section III-C. Sensitivity analyses for crest vertical curve length are found in each of those sections.

H. Sag Vertical Curve Length

1. Current Highway Design and Operational Criteria

The AASHTO Green Book specifies that four different criteria should be considered to some extent in establishing the lengths of sag vertical curves. These are: headlight sight distance, rider comfort, drainage control, and, a rule of thumb for general appearance. Of these, headlight sight distance is considered to be the primary criterion.

The headlight criterion employed in sag vertical curve design is that the vehicle headlights should illuminate the roadway ahead for a length at least equal to the stopping sight distance. Sag vertical curves should be long enough so that, anywhere on the curve, the headlight beam will intersect the pavement at a distance in front of the vehicle that is at least equal to the stopping sight distance. The factors that determine the minimum sag vertical curve length are stopping sight distance, algebraic difference in percent grade, and headlight height. The following equations show how the minimum sag vertical curve length is computed:[33]

When S is less than L_{min}:

$$L_{min} = \frac{AS^2}{200\ (H_h + S\ (\tan B))} \tag{62}$$

When S is greater than L_{min}:

$$L_{min} = \frac{200\ (H_h + S\ (\tan B))}{A} \tag{63}$$

where, L_{min} = Minimum sag vertical curve length (ft)

S = Minimum stopping sight distance (ft)

A = Algebraic difference in percent grade

H_h = Headlight height (ft)

B = divergence angle of light beam from longitudinal axis of vehicle headlight (degrees)

The AASHTO Green Book criteria are based on the stopping sight distance criteria in table 20, a headlight height of 24 in (61 cm), and a divergence angle for the headlight beam of 1° above the centerline of the headlight.

Table 67 shows the minimum sag vertical curve lengths based on the AASHTO criteria.

2. Critique of Highway Design and Operational Criteria

The AASHTO criteria for sag vertical curve length are based on the assumption that vehicle headlights are capable of illuminating the roadway ahead for a distance equal to the stopping sight distance which ranges up to 850 ft (259 m) for a 70-mi/h (113-km/h) design speed, as shown in table 20. However, vehicle headlights are not, in fact, effective in illuminating the roadway over such long distances. The Uniform Vehicle Code, which serves as the model for the traffic laws of many States, specifies that vehicle headlights must be capable of illuminating persons or vehicles at a distance of 450 ft (137 m) for high beams and 150 ft (46 m) for low beams.[83] It is possible that some headlights in use are not, in fact, capable of illuminating the pavement at 450 ft (137 m), even with high beams; it is virtually certain that passenger car headlights will not illuminate the pavement at 850 ft (259 m), as needed for a 70-mi/h (113-km/h) design speed.

The available data on headlight illumination distances imply that approach used by AASHTO to establishing sag vertical curve lengths is inappropriate. It appears that full stopping sight distance for passenger cars at night can be maintained only for design speeds up to about 30 or 40 mi/h (32 to 48 km/h).[33] Thus, the rationale for the AASHTO sag vertical curve criteria needs to be fully reexamined. Apparently, the rationale for use of headlight height in sag vertical curve design dates from an era when single-lane roadways were common and the headlight criterion was developed to assure that approaching drivers could see one another.

The AASHTO criterion for headlight height of 24 in (61 cm) appears to be a reasonable value for design purposes. Until a few years ago, NHTSA required a minimum headlight mounting height of 24 in (61 cm). This has now been lowered to 22 in (56 cm), but probably only a few vehicles have headlights mounted that low. Recent research has suggested, in fact, that 28 in (71 cm) is a more typical headlight height.[33]

Since current headlights are not capable of illuminating the roadway over the full range of stopping sight distances for passenger cars shown in table 20, it is even less likely that truck headlights can provide illumination over the full range of stopping sight distances for trucks, which can be as long as 1,175 ft (358 m) as shown in table 24. However, truck headlights do have the advantage of being mounted higher than passenger car headlights. The Uniform Vehicle Code allows truck headlights to be mounted as high as 54 in (137 cm).[83] Since this is an extreme upper limit for truck headlight height and most truck headlights are lower, a lower value (48 in or 122 cm) was selected to represent truck headlight heights in the sensitivity analysis. The specific value of 48 in (122 cm) for headlight height was selected based on engineering judgment, since no data on the actual distribution of truck headlight heights are available.

Table 67. Minimum sag vertical curve lengths (ft) for passenger cars and trucks.

Algebraic difference in percent grade	Design speed (mi/h) 20	30	40	50	60	70
Passenger Car[a]						
2	40	80	140	220	320	430
4	80	150	280	440	640	860
6	120	220	420	660	950	1,290
8	150	300	560	880	1,270	1,720
10	190	370	690	1,100	1,590	2,150
Truck (Conventional Brake System with 70% Driver Control Efficiency[b]						
2	40	90	190	290	410	570
4	70	180	370	580	830	1,130
6	110	260	560	870	1,240	1,690
8	140	350	740	1,160	1,650	2,260
10	170	430	920	1,450	2,060	2,820
Truck (Conventional Brake System with Best Performance Driver)[c]						
2	30	80	140	210	310	420
4	60	150	270	420	610	830
6	80	230	400	630	910	1,240
8	110	300	540	840	1,210	1,650
10	130	380	670	1,050	1,520	2,060
Truck (Antilock Brake System)[c]						
2	30	60	110	190	250	340
4	60	110	220	370	500	690
6	80	160	330	560	750	1,030
8	110	220	440	740	1,000	1,380
10	130	270	550	920	1,250	1,720

[a] Based on AASHTO sight distance requirements in table 20 and 24-in (61-cm) headlight height.

[b] Based on truck sight distance requirements in table 24 and 48-in (122-cm) headlight height.

[c] Based on truck sight distance requirements in table 23 and 48-in (122-cm) headlight height.

Note: 1 ft = 0.305 m
1 mi = 1.61km
1 in = 2.54 cm

Specific headlight intensity requirements have been established by NHTSA. There is no requirement that headlights provide any light output above the centerline of the headlight, although all headlights in common use in the United States do so. Therefore, the use of a light beam divergence angle of 1° above the centerline of the headlight, as specified by AASHTO, seems reasonable.

3. Sensitivity Analyses

A sensitivity analysis has been conducted to determine whether the current sag vertical curve criteria for passenger cars are adequate for trucks. This analysis was conducted even though the results at higher speeds may be meaningless because of the limited illumination distances of vehicle headlights.

The sensitivity analysis considers whether the longer stopping sight at higher speeds distance requirements of trucks are offset by the higher headlight mounting height. The sight distances used for passenger cars and trucks in this analysis were those given in tables 23 and 24. The braking conditions considered were: a conventional braking system with the 70 percent driver control efficiency, a conventional braking system with the best performing driver (100 percent driver control efficiency), and an antilock braking system. The headlight heights were assumed to be 24 in (61 cm) for passenger cars and 48 in (122 cm) for trucks.

Table 67 presents the minimum sag vertical curve lengths required for each set of passenger car and truck stopping sight distance criteria. The data indicate that longer sag vertical curves are required for a truck with a conventional braking system and the driver with 70 percent control efficiency than are required for a passenger car. However, both truck with a conventional braking system and the best performance driver and the truck with antilock brake system require shorter sag vertical curve lengths than a passenger car in all cases.

4. Summary

Under current AASHTO policies, a change in the criteria sag vertical curve lengths would be appropriate in any situation in which the revised stopping sight distance criteria for trucks in table 24 are used. These criteria are appropriate only for roadways with high truck volumes and only if antilock brake systems for trucks are not required by Government regulation and do not come into nearly universal use.

The model used by AASHTO for sag vertical curve length is itself in need of reconsideration because the rationale for basing sag vertical curve length on headlight beams may be outdated. In addition, the present criteria are flawed because current headlight beams are not capable of illuminating the pavement for the full stopping sight distance needed by high-speed vehicles.

I. Critical Length of Grade

1. Current Highway Design and Operational Criteria

The AASHTO Green Book presents the current warrant for the addition of a truck climbing lane in terms of a "critical length of grade." A climbing lane is not warranted if the grade does not exceed this critical length. If the critical length is exceeded, then a climbing lane is desirable and should be considered. The final decision to install a truck climbing lane may depend on a number of factors, but basically is determined by the reduction in level of service that would occur without the addition. This reduction, in turn, is a function of the traffic volume, the percentage of trucks, the performance capabilities of the trucks, the steepness of the grade, and the length of grade remaining beyond the critical length.

The critical length of grade, itself, is established by the "gradeability" of trucks. Subjectively, the critical length of grade is the "maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed." The Green Book considers the critical length of grade to be dependent on three factors:

1. The weight and power of the representative truck used as the design vehicle, which determine its speed maintenance capabilities on grades.
2. The expected speed of the truck as it enters the critical length portion of the grade.
3. The minimum speed on the grade below which interference to following vehicles is considered unreasonable.

Based on these factors, the AASHTO Green Book defines the critical length of grade as the length of grade that would produce a speed reduction of 10 mi/h (16 km/h) for a 300 lb/hp (0.18 kg/W) truck. The 300 lb/hp (0.18 kg/W) truck is intended for use for average conditions in the United States. The use of a truck with a higher weight-to-power ratio is justified at sites with extremely low-powered or heavily loaded trucks in the traffic stream (e.g., in coal mining regions or near gravel quarries).

2. Critique of the Current Highway Design and Operational Criteria

For the most part, the logical approach followed by the Green Book is well thought out. The procedures to be applied are straightforward and reasonable. Moreover, the AASHTO criteria for factors 2 and 3 also seem reasonable. Factor 1, on the other hand, has been determined using truck performance data that are out of date, and, therefore, needs to be revised. Specific comments on the AASHTO criteria are presented below.

a. Unreasonable Interference with Following Vehicles

The amount of speed reduction used as the criterion for factor 3 in determining the critical length of grade is based on its expected impact on the accident involvement rate of trucks. It is argued, based on known impacts of speed differences between vehicles on accident rates, that any speed difference will increase accident rates to some extent. The amount of this increase that is "reasonable" has been determined through engineering judgment. The Green Book states that the 15-mi/h (24-km/h) speed reduction used in the 1965 Blue Book is no longer reasonable, because the increase in accident involvement rate would be too high.[1,51] The Green Book recommends a 10-mi/h (16-km/h) speed reduction criterion. This decrease in the speed reduction criterion is desirable, because accident rates increase very rapidly with speed difference.

b. Speed at Entrance to the Critical Length of Grade

The Green Book points out, properly, that the speed of trucks on a grade depends, in part, on their speed upon entering the grade. It is reasonable to use the average running speed if the entrance is on level terrain. However, if the upgrade in question is immediately preceded by a previous upgrade, the truck speed may already be depressed, which should be accounted for. Likewise, it is commonly known that truck drivers will accelerate somewhat on a downgrade immediately preceding an upgrade, to get a "running start" at it. In that case, the critical length of grade will be longer than with a level entrance. For the upgrade/upgrade case, the methods used for determining gradeability on a single upgrade can be applied piecewise, to see if the two upgrades, combined, exceed the critical length of grade. For the downgrade/upgrade case, no quantitative information is supplied. A 5-mi/h (8-km/h) increase above the running speed would be a reasonable value, lacking definitive data.

c. Design Vehicle

The Green Book uses a 300-lb/hp (0.18-kg/W) truck as the design vehicle. This weight-to-power ratio is smaller than that used for the AASHO Blue Book; at that time, a 400-lb/hp (0.24-kg/W) truck was typical of a heavy truck.[51] The decision to use the more powerful 300-lb/hp (0.18-kg/W) design vehicle was recommended in a 1978 NCHRP study.[28] Based on the literature at the time, the NCHRP study concluded that the design vehicle should be one that (1) comprises a "large" portion of the vehicle population, and (2) has poor performance characteristics on grades. These criteria appear reasonable.

The data on which the recommendation to use 300 lb/hp (0.18 kg/W) was based were obtained in the early to mid 1970's. Thus, they are now about 15 years old. Yet, most available evidence indicates that truck performance has continued to increase, just as it has since 1949 (see figure 12). The most recently published data are those of Gillespie, which were obtained in 1984.[37] Gillespie reports that the most common heavy trucks, consistently (according to various measures) outperformed the AASHTO 300-lb/hp (0.18-kg/W) design vehicle. Therefore, current policy is overly conservative in that it calls for a shorter critical length of grade than is needed for the current heavy truck population.

Double-trailer trucks do have somewhat poorer performance than single-semitrailer trucks. However, they still perform slightly better than the AASHTO design vehicle. Also, they represent a fairly small fraction of the trucks on the road in most of the United States.

d. Final Climbing Speeds

The most common measure used to quantify truck performance on grades, although not really the correct measure for determining critical length of grade, as will become more evident subsequently, is the final climbing speed. This is the ultimate, slowest speed (the "crawl speed") that the truck would be reduced to if the grade were sufficiently long. It is often reported in the literature, or used in making comparisons between different vehicles. It is a useful measure for examining capacity, for example, on very long grades where trucks are actually reduced to their final climbing speeds. However, the important parameter in determining the critical length of grade is the distance required for the first 10 mi/h (16 km/h) of speed reduction on the grade.

e. Aerodynamics

In order to properly examine the first 10 mi/h (16 km/h) of speed reduction of trucks on grades, it is very important to understand the principles involved. Simply put, the truck engine produces power. That power must overcome several restraining energy-absorbing phenomena:

- Truck acceleration, which produces a gain in kinetic energy.
- Ascending the grade, which produces a gain in potential energy.
- Overcoming friction losses in the truck's drive train.
- Overcoming tire-pavement friction losses.
- Overcoming aerodynamic drag losses.

To a first approximation, the power used to overcome drive train and tire-pavement friction losses are constants, relative to speed. In addition, energy changes due to acceleration or climbing a grade are proportional to vehicle weight. Thus, if we ignore aerodynamic losses, the weight-to-power ratio of the truck is a good measure of its performance, over a range of speeds.

However, the aerodynamic losses of energy are proportional to the square of the speed of the truck. At low speeds (e.g., less than 20 to 30 mi/h [32 to 48 km/h]) these losses are negligible compared to the other losses or to desired gains in kinetic or potential energy. But, at highway speeds, aerodynamic losses become dominant. On level terrain, they are the primary limiting factor in speed attainable for a given engine power. Likewise, upon entering a grade at highway speed, aerodynamic drag plays a major role in determining the rate of speed loss (deceleration), along with the potential energy gains required to ascend the grade.

The importance of aerodynamics to this discussion is simply this: improvement in performance as reflected in reduced weight-to-power ratios for trucks is just part of the story; improved aerodynamic streamlining of trucks must also be considered when performance at (relatively) high speeds is important.

The fuel crisis of the late 1970's spawned a broad range of efforts to improve the fuel efficiency of all vehicles in the United States, including trucks. One of the most obvious changes in this regard was that truckers and truck manufacturers began to treat aerodynamics seriously. Fairings over the tractor cab, to streamline the air flow, are now the norm. Tractors, themselves, are now more aerodynamically efficient. Thus, this fairly recent effort at improved fuel economy has an important impact on the critical length of grade determination.

f. Higher Speeds

Following the energy crises of the late 1970's, the speed limits on the United States' highways were set at 55 mi/h (89 km/h), where they remained for a number of years. Only recently has Congress allowed the States to increase the limits to 65 mi/h (105 km/h) on rural Interstate roads. This allowance is now expanding, gradually, to other rural routes. The impact of this trend is to emphasize further the importance of aerodynamic drag on the total performance of trucks and their ability to maintain speeds on grades.

3. Sensitivity Analyses

There is no simple formula for critical length of grade for which an explicit sensitivity analysis can be performed. The key feature in setting realistic criteria for critical length of grade is the selection of a value for weight-to-power ratio that is representative of the truck population that is currently on the road and will be on the road in the future.

Figure 12 demonstrates that truck weight-to-power ratios have been decreasing steadily for four decades. Recent research suggests that these trends are continuing. Unfortunately, the most recent study on this issue by Gillespie did not provide results in the form of a distribution of weight-to-power ratios for the trucks observed. Therefore, the Gillespie data were reanalyzed in the present study and the results are reported in appendix D in volume II.

The reanalysis of the Gillespie data found that in the mid 1980's the 87.5th percentile weight-to-power ratio for combination trucks was approximately 250 lb/hp (0.15 kg/W). This finding agrees well with the results of another analysis performed in the present study. This analysis applied a truck performance model developed by St. John and Kobett to Caltrans data collected in the late 1970's and updated in 1983 and 1984.[30,34] This analysis found that the weight-to-power ratio of the 93.5th percentile truck was 245 lb/hp (0.15 kg/W).

Based on the truck population currently (or, at least, recently) on the road, it is recommended that the weight-to-power ratio used to determine critical length of grade be reduced to 250 lb/hr (0.15 kg/W). However, acting on this recommendation should be deferred until the resolution of the so-called "Turner truck" proposal. This proposal by former FHWA administrator Francis C. Turner could allow truck gross weights as high as 150,000 lb (68,000 kg) as long as the number of axles and axle spacings of the truck meet an established bridge formula. This proposal, if implemented, could possibly increase the weight-to-power ratios enough to warrant retention of the current 300-lb/hp (0.18-kg/W) criterion. No reliable estimates are available of the tractor horsepowers that would be used with "Turner trucks." However, historical data show that truck operators are unlikely to choose tractors that leave their trucks substantially underpowered. Tractors up to 500 hp (373 kW) are currently available in the U.S. market and it is likely that more powerful tractors would be developed if there was a demand for them. A 150,000-lb (68,000-kg) truck with a 500-hp (373-kW) tractor would operate at a weight-to-power ratio of 300 lb/hp (0.18 kg/W). If a 600-hp (447-kW) tractor were marketed in the future, a fully loaded 150,000-lb (68,000-kg) truck could operate at a weight-to-power ratio of 250 lb/hp (0.15 kg/W), equivalent to the 87.5th percentile value in the current truck population.

4. Recommended Revisions to Highway Design and Operational Criteria

An analysis based on two sets of field data collected in the early to mid-1980's indicates that, based on the current truck population, the weight-to-power ratio for the design truck used in the AASHTO critical length of grade criteria should be reduced from 300 to 250 lb/hp (0.18 to 0.15 kg/W). The 10-mi/h (16-km/h) speed reduction criterion for critical length of grade should be retained.

Implementation of this recommendation should be deferred until the current "Turner truck" proposal is resolved. This proposal might necessitate retention of the current 300 lb/hp (0.18 kg/W) criterion, although 150,000-lb (68,000-kg) trucks could operate at 250 lb/hp (0.15 kg/W) if market forces create demand for more powerful tractors.

J. Lane Width

1. Highway Design and Operational Criteria

The AASHTO Green Book encourages the use of 12-ft (3.7-m) lanes for all but the lowest volume highways. In particular, on rural arterials, lane widths less than 12 ft (3.7 m) are normally used only for roads with design speeds under 60 mi/h (92 km/h) and average daily traffic (ADT) under 400 veh/day. Under restrictive or special conditions, 11-ft (3.4-m) lanes may be acceptable. For urban arterials, the AASHTO Green Book states that 10-ft (3.1 m) lanes are used only in highly restricted areas having little or no truck traffic. However, both 11- and 12-ft (3.4- and 3.7-m) lane widths are used extensively on urban arterials.

The AASHTO Green Book does encourage wider lanes to accommodate trucks on some turning roadways at intersections and some horizontal curves. These issues are discussed in sections III-E and III-L, respectively.

2. Critique of Design and Operational Criteria

The lane width criteria in the AASHTO Green Book were established without reference to any explicit vehicle width specification. However, it is implicit in the criteria that the need for 11-ft and 12-ft (3.4- and 3.7-m) lanes is based on the consideration of truck width. This is particularly important in light of the 1982 Surface Transportation Assistance Act (STAA) which permits 102-in (259 cm) wide trucks to operate on an extensive system of designated highways. Previously, only 96-in (244 cm) wide trucks were permitted by most States.

Two studies have addressed the operational effects of wider vehicles and the implications of these effects for highway design. A joint NHTSA-FHWA assessment conducted in 1973 compared the operational effects of 96- vs. 102-in (244- vs. 259-cm) wide buses on two-lane, four-lane, six-lane, and eight-lane highways based on research reported by in the literature.[85,86] This research found no effect of bus width on the lateral placement of adjacent cars regardless of highway type and ambient wind conditions. There was a shift in the lateral position of cars by 12 to 18 in (30 to 46 cm) when a bus was present, but the magnitude of this shift did not vary between 96- and 102-in (244- and 259-cm) wide buses.

A 1982 FHWA study of the effects of truck width on the positions of adjacent vehicles found no adverse effects of increased truck width either in passing maneuvers or at narrow bridges.[87] The passing maneuver studies were conducted on a two-lane highway with lane widths that varied from 10.5 to 12 ft (3.2 to 3.7 m). Vehicle widths of 96, 102, 108, and 114 in (244, 259, 274, and 290 cm) were varied by changing the width of a fabricated wood and aluminum box on the trailer. The lateral position of the passing vehicle moved further to the left as the truck width increased, but there was effect of truck widths on shoulder encroachments in passing maneuvers, which were observed consistently in about 6 percent of the passes. In studies at a narrow bridge on a two-lane highway with 11.5-ft (3.5-m) lanes, there was no effect of truck width on the speed or lateral placement of oncoming vehicles.

A 1987 FHWA study found that lane widths affect both single vehicle accidents (such as fixed object, rollover, and run-off-road accidents) and multivehicle accidents (such as head-on and sideswipe accidents).[88] Further, a predictive model for two-lane highways developed in this study shows that for all vehicles, a 1-ft (0.3 m) increase in lane width results in a 12 percent reduction in the types of accidents mentioned above. Widening by 2 ft (0.6 m) produces a 23 percent reduction and 4 ft (0.6 m) provides a 40 percent reduction in the previously mentioned accident types. This model is based upon lane widths between 8 and 12 ft (2.4 and 3.7 m). However, there is no indication in this study that this safety effect relates directly to truck widths.

3. Sensitivity Analysis

No sensitivity analyses were conducted because there is no explicit relationship of truck width to lane width requirements.

4. Summary

There is no indication in the literature that lanes widths of 11 and 12 ft (3.4 and 3.7 m), which are normally used under existing design criteria on roads where substantial truck volumes are present, are not adequate for 102-in (259-cm) trucks. There does not appear to be any justification for considering a change in current lane width design criteria based on truck considerations.

K. Horizontal Curve Radius and Superelevation

This section of the report examines the role of truck considerations in the design of horizontal curves. Pavement widening on horizontal curves is addressed in the next section.

1. Current Design and Operational Criteria

The current design criteria for horizontal curves are established in the AASHTO Green Book. Under the AASHTO policy, a vehicle on a horizontal curve is represented as a point mass. From the basic laws of physics, the lateral acceleration of a point mass traveling at constant speed on a circular path can be represented by the relationship:

$$a = \frac{V^2}{15R} \tag{64}$$

where: a = lateral acceleration (g)

V = vehicle speed (mi/h)

R = radius of curve (ft)

The lateral acceleration experienced by the vehicle is expressed in units of the acceleration of gravity (g) which are equal to 32.2 ft/s² (9.8 m/s²). On a superelevated curve, the superelevation offsets a portion of the lateral acceleration, such that:

$$a_{net} = \frac{V^2}{15R} - e \tag{65}$$

where: a_{net} = unbalanced portion of lateral acceleration (g)

e = superelevation (ft/ft)

The unbalanced portion of the lateral acceleration of the vehicle is a measure of the forces acting on the vehicle that tend to make it skid off the road or overturn. The side frictional demand of the vehicle is mathematically equivalent to the unbalanced lateral acceleration (a_{net}). For this reason, equation (65) appears in the AASHTO Green Book in the form:

$$f = \frac{V^2}{15R} - e \tag{66}$$

where: f = side friction demand

The tendency of the vehicle to skid off the road must be resisted by tire/pavement friction. The vehicle will skid off the road unless the tire/pavement friction coefficient exceeds the side friction demand. However, it is also critical for safe vehicle operations that vehicles not roll over on horizontal curves. The tendency of the vehicle to overturn must be resisted by the roll stability of the vehicle. The vehicle will roll over unless the rollover threshold of the vehicle exceeds the unbalanced lateral acceleration (a_{net}).

a. Selection of Radius and Superelevation

The objective of AASHTO criteria for horizontal curve design is to select the radius and superelevation so that the unbalanced lateral acceleration is kept within tolerable limits. AASHTO policy limits the unbalanced lateral acceleration for horizontal curves to a maximum of 0.17 g at 20 mi/h (32 km/h) decreasing to a maximum of 0.10 g at 70 mi/h (113 km/h). This limitation is based on the results of research performed in 1936 through 1949 that established 0.17 g as the maximum unbalanced lateral acceleration at which drivers felt comfortable. Thus, it is important to note that these AASHTO criteria are based on maintaining comfort levels for passenger car drivers. The AASHTO criteria are not based explicitly on estimates of available tire/pavement friction levels or vehicle rollover thresholds, although it was assumed implicitly that available friction levels and rollover thresholds were higher than the specified driver comfort levels.

The AASHTO Green Book provides design charts for maximum superelevation rates (e_{max}) from 0.04 to 0.10. Highway agencies have established their own policies concerning the maximum superelevation rate that will be used on horizontal curves under their jurisdiction. Most highway agencies use maximum superelevation rates of either 0.06 or 0.08; States that experience snow and ice conditions typically use lower superelevation rates. For any particular maximum superelevation rate and maximum side friction demand, the minimum radius of curvature can be determined as:

$$R_{min} = \frac{V_d^2}{15\ (e_{max} + f_{max})} \tag{67}$$

where: R_{min} = minimum radius of curvature (ft)

V_d = design speed of curve (mi/h)

e_{max} = specified maximum superelevation rate (ft/ft)

f_{max} = specified maximum side friction demand

Table 68 presents the minimum radius of curvature for specific combinations of maximum superelevation rate and maximum side friction demand considered in AASHTO policy.

Table 68. AASHTO criteria for maximum degree of curve and minimum radius for horizontal curves on rural highways and high-speed urban streets.[1]

Design Speed (mph)	Maximum e	Maximum f	Total (e + f)	Maximum Degree of Curve	Rounded Maximum Degree of Curve	Minimum Radius (ft)
20	.04	.17	.21	44.97	45.0	127
30	.04	.16	.20	19.04	19.0	302
40	.04	.15	.19	10.17	10.0	573
50	.04	.14	.18	6.17	6.0	955
60	.04	.12	.16	3.81	3.75	1,528
20	.06	.17	.23	49.25	49.25	116
30	.06	.16	.22	20.94	21.0	273
40	.06	.15	.21	11.24	11.25	509
50	.06	.14	.20	6.85	6.75	849
60	.06	.12	.18	4.28	4.25	1,348
65	.06	.11	.17	3.45	3.5	1,637
70	.06	.10	.16	2.80	2.75	2,083
20	.08	.17	.25	53.54	53.5	107
30	.08	.16	.24	22.84	22.75	252
40	.08	.15	.23	12.31	12.25	468
50	.08	.14	.22	7.54	7.5	764
60	.08	.12	.20	4.76	4.75	1,206
65	.08	.11	.19	3.85	3.75	1,528
70	.08	.10	.18	3.15	3.0	1,910
20	.10	.17	.27	57.82	58.0	99
30	.10	.16	.26	24.75	24.75	231
40	.10	.15	.25	13.38	13.25	432
50	.10	.14	.24	8.22	8.25	694
60	.10	.12	.22	5.23	5.25	1,091
65	.10	.11	.21	4.28	4.25	1,348
70	.10	.10	.20	3.50	3.5	1,637

NOTE: In recognition of safety considerations, use of e_{max} = 0.04 should be limited to urban conditions.

Note: 1 mi = 1.61 km
1 ft = 0.305 m

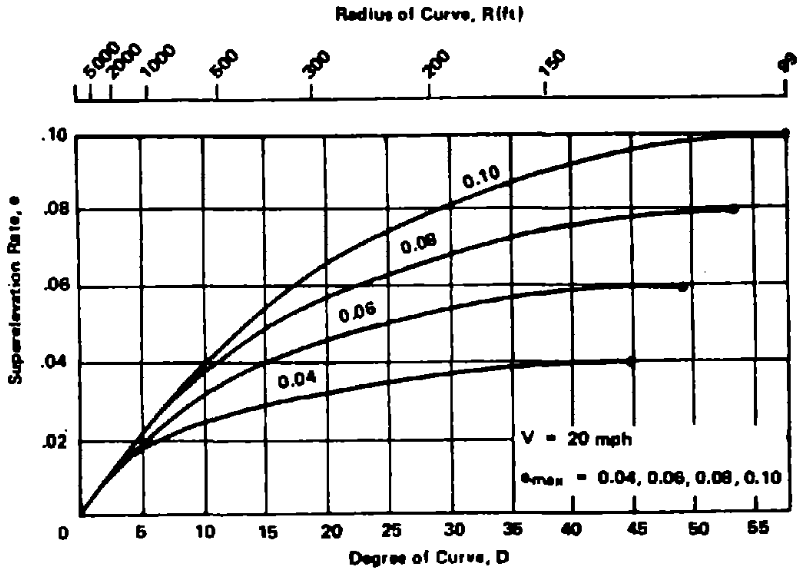
The radius of a horizontal curve can also be expressed as the degree of curvature, defined as the change of heading in degrees per 100 ft (30 m) of curve length. The degree of curvature is computed as:

$$D = \frac{5,730}{R} \tag{68}$$

where: D = degree of curvature (degrees/100 ft).

In the design of a horizontal curve under AASHTO policy, the first major decision is to select its radius of curvature. Next, the selected radius is checked to assure that it is not less than R_{min} for the design speed of the highway. Finally, if the selected radius is greater than R_{min}, a superelevation less than e_{max} is selected using tables III-8 through III-11 of the

AASHTO Green Book. Figure III-9 of the AASHTO Green Book, presented here as figure 43, summarizes the superelevation rates used for curves with radii greater than R_{min}.



Note: In recognition of safety considerations, use of e_{max} = 0.04 should be limited to urban conditions.

1 ft = 0.305 m
1 mi = 1.61 km

Figure 43. AASHTO criteria for superelevation rates of horizontal curves as a function of radius and maximum superelevation rate.[1]

b. Transition Design

Most horizontal curves are circular curves that directly adjoin tangent roadway sections at either end with no transition curve. Thus, a vehicle entering a curve theoretically encounters an instantaneous increase in lateral acceleration from a minimal level of the tangent section to the full lateral acceleration required to track the particular curve. The opposite occurs as a vehicle leaves a horizontal curve. In fact, there is a gradual rather than an instantaneous change in lateral acceleration, because drivers steer a spiral or transition path as they enter or leave a horizontal curve. The design of the superelevation transition section is used to partially offset the changes in lateral acceleration that do occur. First, a superelevation runout section is used on the tangent section to remove the adverse crown slope. Next, a superelevation runoff section is provided in which the pavement is rotated around its inside edge to attain the full required superelevation; typical design practice is to place two-thirds of the superelevation runoff on the tangent approach and one-third on the curve. Table III-14 in the AASHTO Green Book presents the required length for superelevation runoff on two-lane pavements.

The AASHTO Green Book encourages the use of spiral transition curves to provide a smooth transition between tangents and circular curves. In a spiral transition curve, the degree of curvature varies linearly from zero at the tangent end to the degree of the circular arc at the circular curve end. The length of the spiral transition curve can be made the same as the superelevation runoff, so that the degree of curvature and pavement cross-slope change together.

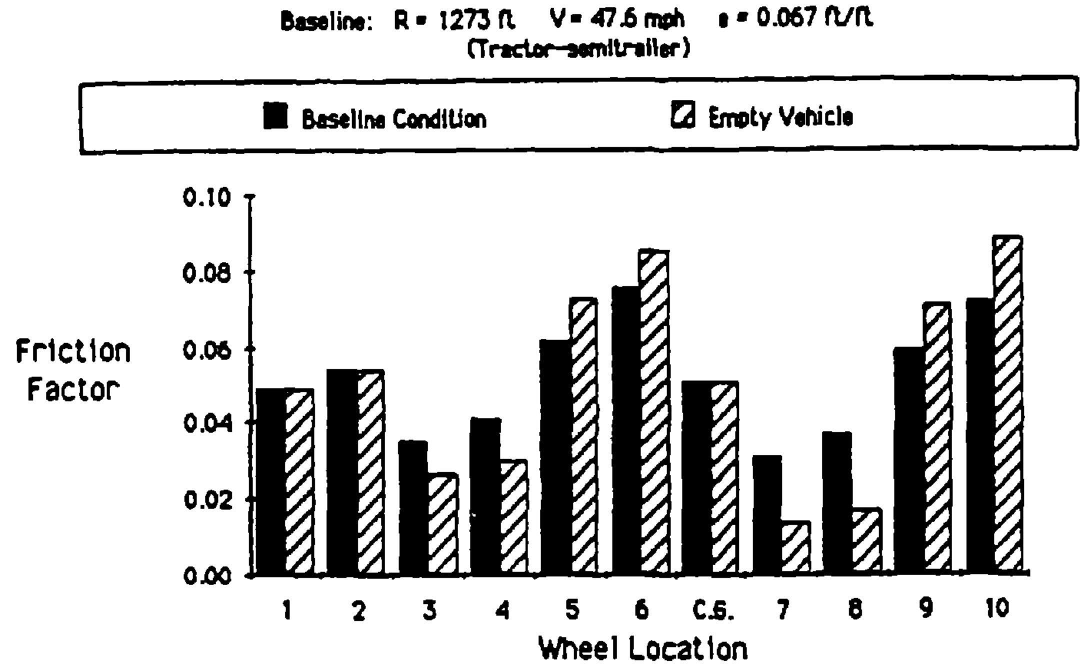
2. Critique of Design and Operational Policy

a. Consideration of Friction Demand

The point mass representation of a vehicle that forms the basis for equations (64), (65), and (66) is not based on any particular set of vehicle characteristics and is theoretically as applicable to trucks as to passenger cars. However, in light of the differences between passenger cars and trucks in size, number of tires, tire characteristics, and suspension characteristics, the suitability of the equations for trucks was recently reexamined.

A 1985 FHWA study found that, since the basic laws of physics apply to both passenger cars and trucks, the point mass representation in equation (66) can be used to determine the net side friction demand of both passenger cars and trucks.[89] However, they found that while the friction demands at the four tires of a passenger car are approximately equal, the friction demands at the various tires of a tractor-trailer truck vary widely, as illustrated in figure 44. The net result of this tire-to-tire variation in friction demand is that trucks typically demand approximately 10 percent higher side friction than passenger cars. We have termed this higher side friction demand the "effective side friction demand" of trucks.

The point mass representation of a vehicle has another weakness, however, that applies to both passenger cars and trucks. Equation (66) is based on the assumption that vehicles traverse curves following a path of constant radius equal to the radius of the curve. However, field studies have shown that all vehicles oversteer at some point on a horizontal curve. At the point of oversteering, the vehicle is following a path radius that is less than the radius of the curve.[90] Thus, at some point on each curve, the friction demand of each vehicle will be slightly higher than suggested by Equation (66). Oversteering by passenger cars is not considered in the AASHTO design policy for horizontal curves, but it is probably not critical because the AASHTO maximum lateral acceleration requirements are based on driver comfort levels rather than the available pavement friction. No data are available on the amount of oversteering by trucks relative to passenger cars.



Note: The wheel locations are for a 5-axle tractor-semitrailer and start at the front axle with wheel location number 1. Odd numbers represent the outside wheels on the turn.
1 ft = 0.305 m

Figure 44. Example of variation in side friction demand between wheels of a truck on a horizontal curve.[89]

b. Consideration of Rollover Threshold

As demonstrated above, AASHTO criteria for horizontal curve design do not explicitly consider vehicle rollover thresholds. The rollover threshold for passenger cars may be as high as 1.2 g, so a passenger car will normally skid off a road long before it would roll over. Thus, the consideration of rollover threshold is not critical for passenger cars. However, tractor-trailer trucks have relatively high centers-of-gravity and consequently tend to have low rollover thresholds. Furthermore, due to suspension characteristics, the rollover threshold of tractor-trailer trucks is substantially less than it would be if a truck were a rigid body.

Recent research, summarized in section II-H of this report, has determined the rollover thresholds of a number of common trucks with typical loading configurations. Some trucks with rollover thresholds as low as 0.30 g are found on the road. Since AASHTO design policy permits lateral acceleration as large as 0.17 g, the margin of safety for trucks with low rollover thresholds on some horizontal curves is not great. Furthermore, as discussed above, oversteer will generally result in a lateral acceleration greater than f_{max} at some point on the curve for vehicles traveling at the design speed.

As an example of truck operations on horizontal curves, figure 45 presents the distribution of nominal side friction demand for trucks from combined data on four curves in the Chicago area as part of a NHTSA study.[91] The radii of the four curves range from 220 to 840 ft (67 to 256 m) and the superelevations range from 0.02 to 0.088. The distribution in figure 45 was developed by measuring truck speeds on the curve and calculating the lateral acceleration for each truck from the known radius and superelevation using equation (65).

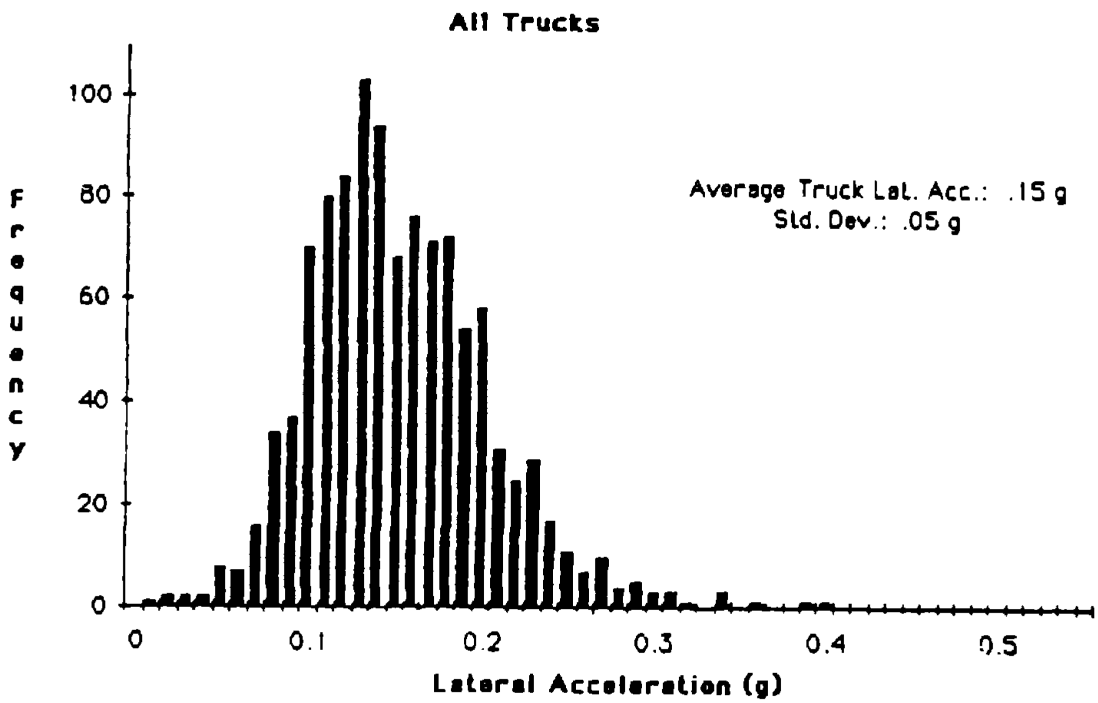


Figure 45. Nominal lateral accelerations of trucks based on their observed speeds on selected horizontal curves in the Chicago area.[91]

The figure illustrates that trucks generating lateral accelerations above 0.30 g are observed, and the lateral accelerations for some trucks range as high as 0.40 g. No generalizations should be drawn from these data, since they represent only four particular horizontal curves, but they do illustrate that levels of side friction demand that could produce rollovers for some trucks can occur.

3. Sensitivity Analyses

Sensitivity analyses have been conducted to determine whether the existing horizontal curve design criteria are adequate to accommodate trucks. The adequacy of the existing criteria was evaluated with respect to both their ability to keep vehicles from skidding off the road and their ability to keep vehicles from rolling over. These sensitivity analyses involved explicit comparisons between the margins of safety against skidding and rollover for passenger cars and trucks. Special emphasis is placed on concern about vehicles traveling faster than the design speed, particularly on freeway ramps.

a. Margin of Safety Against Skidding

Current design criteria for horizontal curves are intended to keep vehicles from skidding off the road on wet pavements. The criteria are based on the standard curve formula, which provides that a portion of the lateral acceleration developed by the vehicle will be resisted by superelevation and the remainder by tire-pavement friction. Table 69 shows that current criteria provide a margin of safety of 0.31 to 0.41 g against a passenger car skidding off the road on a minimum radius curve on wet pavement when traveling at the design speed. The margin of safety is the magnitude of the additional lateral acceleration that the vehicle could undergo without skidding. The pavement friction levels used in the table are the locked-wheel friction levels assumed in current AASHTO stopping sight distance criteria multiplied by 1.45 to adjust them to peak friction coefficients, which are more appropriate for cornering maneuvers. These friction coefficients represent low, but not extreme values, of tire-pavement friction for passenger cars on wet pavements.

Tire-pavement friction on a given pavement is lower for truck tires than for passenger car tires. NCHRP Report 270 estimates that truck tires have coefficients of friction that are only about 70 percent of those of passenger car tires.[12] In addition, the 1985 FHWA study discussed above has shown that trucks generate friction demands approximately 10 percent higher than passenger cars when traversing a curve.[89] Thus, table 69 shows that the margin of safety against a truck skidding off the road on a wet pavement is less than for a passenger car. The margin of safety against skidding for a truck traveling at the design speed on a minimum radius curve on a wet-pavement ranges from 0.17 to 0.22 g.

On dry pavements, tire-pavement friction is much higher than on wet pavement. Locked wheel pavement friction coefficients of 0.65 or [illegible]e are typical for passenger cars on dry surfaces, as shown in figure [illegible] of the AASHTO Green Book. Thus, peak friction levels would be even highe[illegible] by a factor of 1.45. Peak friction levels for trucks were assumed to be 56 percent of the values for passenger cars. As shown in table 71, the margin of safety for both passenger cars and trucks on dry surfaces is much higher than on wet surfaces.

Table 69. Margins of safety against skidding on horizontal curves.

		Passenger car						Truck					
Design speed (mi/h)	Maximum super-elevation e	Maximum tolerable lateral acceleration (g)	Maximum demand f	Minimum radius (ft)	Available f (wet)	Margin of safety (wet)	Margin of safety (dry)	Maximum tolerable lateral acceleration (g)	Minimum radius (ft)	Maximum demand f	Truck available f (wet)	Margin of safety (wet)	Margin of safety (dry)
20	0.04	0.17	0.17	127	0.58	0.41	0.77	0.17	127	0.19	0.41	0.22	0.47
30	0.04	0.16	0.16	302	0.51	0.35	0.78	0.16	302	0.18	0.36	0.18	0.48
40	0.04	0.15	0.15	573	0.46	0.31	0.79	0.15	573	0.17	0.32	0.16	0.49
50	0.04	0.14	0.14	955	0.44	0.30	0.80	0.14	955	0.15	0.30	0.15	0.51
60	0.04	0.12	0.12	1,528	0.42	0.30	0.82	0.12	1,528	0.13	0.29	0.16	0.53
20	0.06	0.17	0.17	116	0.58	0.41	0.77	0.17	116	0.19	0.41	0.22	0.47
30	0.06	0.16	0.16	273	0.51	0.35	0.78	0.16	273	0.18	0.36	0.18	0.48
40	0.06	0.15	0.15	509	0.46	0.31	0.79	0.15	509	0.17	0.32	0.16	0.49
50	0.06	0.14	0.14	849	0.44	0.30	0.80	0.14	849	0.15	0.30	0.15	0.51
60	0.06	0.12	0.12	1,348	0.42	0.30	0.82	0.12	1,348	0.13	0.29	0.16	0.53
70	0.06	0.10	0.10	2,083	0.41	0.31	0.84	0.10	2,083	0.11	0.28	0.17	0.55
20	0.08	0.17	0.17	107	0.58	0.41	0.77	0.17	107	0.19	0.41	0.22	0.47
30	0.08	0.16	0.16	252	0.51	0.35	0.78	0.16	252	0.18	0.36	0.18	0.48
40	0.08	0.15	0.15	468	0.46	0.31	0.79	0.15	468	0.17	0.32	0.16	0.49
50	0.08	0.14	0.14	764	0.44	0.30	0.80	0.14	764	0.15	0.30	0.15	0.51
60	0.08	0.12	0.12	1,206	0.42	0.30	0.82	0.12	1,206	0.13	0.29	0.16	0.53
70	0.08	0.10	0.10	1,910	0.41	0.31	0.84	0.10	1,910	0.11	0.28	0.17	0.55
20	0.10	0.17	0.17	99	0.58	0.41	0.77	0.17	99	0.19	0.41	0.22	0.47
30	0.10	0.16	0.16	231	0.51	0.35	0.78	0.16	231	0.18	0.36	0.18	0.48
40	0.10	0.15	0.15	432	0.46	0.31	0.79	0.15	432	0.17	0.32	0.16	0.49
50	0.10	0.14	0.14	694	0.44	0.30	0.80	0.14	594	0.15	0.30	0.15	0.51
60	0.10	0.12	0.12	1,091	0.42	0.30	0.82	0.12	1,091	0.13	0.29	0.16	0.53
70	0.10	0.10	0.10	1,637	0.41	0.31	0.84	0.10	1,637	0.11	0.28	0.17	0.55

Note: 1 mi = 1.61 km
1 ft = 0.305 m

A simple example will show how the margin of safety against skidding is calculated using the data in the first row of table 69. This row represents a horizontal curve with a design speed of 20 mi/h (32 km/h) and a maximum superelevation of 0.04. Under AASHTO policy, a horizontal curve with a design speed of 20 mi/h (32 km/h) can be designed with a maximum tolerable lateral acceleration of 0.17 g. An equivalent statement is that the maximum side friction demand for a vehicle traveling at the design speed on a curve with maximum superelevation is 0.17 g. The minimum radius of curvature for this situation can be determined from equation (67) as:

$$R_{min} = \frac{(20)^2}{15(0.04+0.17)} = 127 \text{ ft } (39 \text{ m})$$

The assumed pavement friction coefficient at 20 mi/h (32 km/h) for a passenger car tire on a wet pavement is specified in AASHTO policy as 0.40, as shown in table 6. The peak friction coefficient available for cornering on a wet pavement is computed as:

$$0.40\ (1.45) = 0.58$$

A peak friction coefficient of 0.58 means that a vehicle can generate up to 0.58 g of unbalanced laterial acceleration without skidding. Therefore, the margin of safety against skidding for a passenger car on a wet pavement traveling at the design speed under assumed design conditions can be computed as the difference between the maximum lateral acceleration that can be developed without exceeding the available friction (0.58 g) and the friction demand (0.17 g):

$$0.58 - 0.17 = 0.41$$

The pavement friction coefficient under dry conditions was estimated as 0.65, as described above. Under dry conditions, the peak friction available for cornering is computed as:

$$0.65\ (1.45) = 0.94$$

Therefore, the margin of safety against skidding under dry conditions is:

$$0.94 - 0.17 = 0.77$$

The calculations of the margin of safety against skidding for a truck are similar. As discussed above, the maximum demand friction for a truck is 10 percent higher than for a passenger car based on the results of a 1985 FHWA study.[89] Thus, when a truck is traversing a horizontal curve at the design speed under design conditions at the maximum tolerable lateral acceleration of 0.17 g, the effective maximum friction demand is:

$$0.17\ (1.1) = 0.19$$

Since research has shown that truck tires can generate only about 70 percent of the friction of passenger car tires, the peak friction available under wet conditions for a truck is:

$$0.58\ (0.70) = 0.41$$

and the margin of safety under wet conditions is:

$$0.41 - 0.19 = 0.22$$

Similarly, under dry conditions, the available peak friction for a truck tire is:

$$0.94\ (0.70) = 0.66$$

and the margin of safety under dry conditions is:

$$0.66 - 0.19 = 0.48$$

The margins of safety for trucks in table 69 are large enough to provide safe truck operations if there are no major deviations from the basic assumptions used in horizontal curve design. The effects of deviations from the basic assumptions are considered below.

b. Margin of Safety Against Rollover

Table 70 presents an analysis of the margin of safety against rollover provided by current horizontal curve design criteria. The margin of safety is the magnitude of the additional lateral acceleration that the vehicle could undergo without rolling over. The table shows the rollover margin of safety

Table 70. Margins of safety against rollover on horizontal curves.

		Passenger car			Truck					
							Rollover margin of safety			
Design speed (mi/h)	Maximum *e*	Maximum tolerable lateral acceleration	Minimum radius (ft)	Rollover margin of safety RT = 1.20 g	Maximum tolerable lateral acceleration	Minimum radius (ft)	RT = 0.27 g	RT = 0.30 g	RT = 0.35 g	RT = 0.40 g
20	0.04	0.17	127	1.03	0.17	127	0.10	0.13	0.18	0.23
30	0.04	0.16	302	1.04	0.16	302	0.11	0.14	0.19	0.24
40	0.04	0.15	573	1.05	0.15	573	0.12	0.15	0.20	0.25
50	0.04	0.14	955	1.06	0.14	955	0.13	0.16	0.21	0.26
60	0.04	0.12	1,528	1.08	0.12	1,528	0.15	0.18	0.23	0.28
20	0.06	0.17	116	1.03	0.17	116	0.10	0.13	0.18	0.23
30	0.06	0.16	273	1.04	0.16	273	0.11	0.14	0.19	0.24
40	0.06	0.15	509	1.05	0.15	509	0.12	0.15	0.20	0.25
50	0.06	0.14	849	1.06	0.14	849	0.13	0.16	0.21	0.26
60	0.06	0.12	1,348	1.08	0.12	1,348	0.15	0.18	0.23	0.28
70	0.06	0.10	2,083	1.10	0.10	2,083	0.17	0.20	0.25	0.30
20	0.08	0.17	107	1.03	0.17	107	0.10	0.13	0.18	0.23
30	0.08	0.16	252	1.04	0.16	252	0.11	0.14	0.19	0.24
40	0.08	0.15	468	1.05	0.15	468	0.12	0.15	0.20	0.25
50	0.08	0.14	746	1.06	0.14	764	0.13	0.16	0.21	0.26
60	0.08	0.12	1,206	1.08	0.12	1,206	0.15	0.18	0.23	0.28
70	0.08	0.10	1,910	1.10	0.10	1,910	0.17	0.20	0.25	0.30
20	0.10	0.17	99	1.03	0.17	99	0.10	0.13	0.18	0.23
30	0.10	0.16	231	1.04	0.16	231	0.11	0.14	0.19	0.24
40	0.10	0.15	432	1.05	1.15	432	0.12	0.15	0.20	0.25
50	0.10	0.14	694	1.06	0.14	694	0.13	0.16	0.21	0.26
60	0.10	0.12	1,091	1.08	0.12	1,091	0.15	0.18	0.23	0.28
70	0.10	0.10	1,637	1.10	0.10	1,637	0.17	0.20	0.25	0.30

Note: 1 mi = 1.61 km
1 ft = 0.305 m

in units of the acceleration of gravity (g) for passenger cars with rollover thresholds of 1.20 g and for trucks with rollover thresholds from 0.27 to 0.40 g.

The margin of safety against rollover for passenger cars traveling at the design speed ranges from 1.03 to 1.10 g. At all design speeds, the margin of safety against rollover for a passenger car is much higher than the margin of safety against skidding on either a wet or dry pavement. Thus, rollover is not a major concern for passenger cars because, unless they collide with another vehicle or object, passenger cars will skid rather than roll over. In contrast to the related issue of skidding off the road, the margin of safety against rollover is not dependent on whether the pavement is wet or dry.

Section II-H of this report establishes that a conservative value of truck rollover threshold appropriate for use in design is 0.30 g. The margin of safety for a truck with a rollover threshold of 0.30 g ranges from 0.13 to 0.20 g. This margin of safety is adequate to prevent rollover for trucks traveling at or below the design speed. It should be noted that the margin of safety against rollover increases with increasing design speed, while the margin of safety against skidding decreases.

Comparison of tables 69 and 70 indicates that rollover is a particular concern for trucks. Under the assumed design conditions for horizontal curves, a truck will roll over before it will skid on a dry pavement. Under the assumed design conditions on a wet pavement, a truck will roll over before it skids at design speeds of 40 mi/h (64 km/h) and below; above that speed, a truck will skid before it rolls over. The effects of deviations from the basic assumptions are considered below.

c. Deviations from Assumed Design Conditions

The margins of safety against skidding and rollover are a measure of the extent to which real-world drivers, vehicles, and highways can deviate from the assumed conditions without resulting in a skid or a rollover. Deviations from assumed conditions that can increase the likelihood of skidding include:

- Vehicles traveling faster than the design speed.
- Vehicles turning more sharply than the curve radius.
- Lower pavement friction than assumed by AASHTO.
- Poorer tires than assumed by AASHTO.

Traveling faster than the design speed and turning more sharply than the curve radius would also increase the likelihood of rollovers. In addition, the likelihood of a rollover would also be increased for a truck with a rollover threshold less than the assumed value of 0.30 g.

It would seem logical that the practice of providing less than full superelevation at the point of curvature (PC) would also increase the likelihood of rollovers, but this is not always the case. Horizontal curves without spiral transitions are typically designed with 2/3 of the superelevation runoff on the tangent in advance of the PC and 1/3 of the superelevation runoff on the curve itself. Thus, only 2/3 of the design superelevation is available at the PC and this lack of full superelevation at the PC would appear to have the potential to offset up to approximately 0.03 g of the available margin of safety. However, AASHTO policy assumes and field and simulation studies (for passenger cars) confirm that even on horizontal curves without spiral transitions, drivers tend to steer a spiral path. Thus, where maximum superelevation is not available, the driver is usually not steering a minimum radius path.

Computer simulation studies of trucks traversing horizontal curves reported in appendix B of volume II found that developing full superelevation on the tangent approach to a conventional circular curve actually developed slightly more lateral acceleration than development of superelevation with the 2/3-1/3 rule. While the difference in lateral acceleration is small--at most 0.03 g--it is in the wrong direction, so development of full superelevation on the tangent is not a desirable approach to reducing truck rollovers. The same study found a small decrease in lateral acceleration--typically less than 0.01 g--when spiral transitions were used to develop the superelevation. Thus, the use of spiral transitions is desirable but, because of the small reduction in lateral acceleration, the use of spirals is unlikely to provide a major reduction in rollover accidents.

Field data for passenger cars (and simulation results for trucks obtained in this study) show that vehicles traversing a curve do not precisely follow the curve. Thus, while the path may have a larger radius than the curve at the PC, it will also have a smaller radius than the curve at some point in the curve. Simulation results show that the maximum lateral acceleration occurs several hundred feet after entering a curve. However, simulation results also show that the maximum excursion of lateral acceleration above the value obtained from the standard curve formula is approximately 0.02 g, which would offset a small portion of the margins of safety against rolling and skidding. Field studies for passenger cars suggest that this is a reasonable average value, but more extreme values can occur. Truck drivers may have lower excursions of lateral acceleration than passenger car drivers, but there are no data on this point.

The AASHTO criteria for tire-pavement friction are based on a poor, wet pavement and (apparently) on worn tires. Table 71 has provided an adjustment to these values for the differences between passenger cars and trucks. The assumptions appear to be conservative for design purposes. In fact, an interesting aspect of this factor discussed below is what happens when the likelihood of skidding is reduced because tire pavement-friction is higher than the design value.

The review of the potential for safety problems created by derivations from the design assumptions indicates that traveling faster than the design speed of the curve is the single greatest concern. This is a particular concern on freeway ramps for two reasons. First, freeway ramps generally have lower design speeds than mainline roadways, which means that they have lower margins of safety against rollover (but higher margins of safety against skidding). Second, traveling faster than the design speed is especially likely on off-ramps, where vehicles traveling at higher speeds enter the ramp from the mainline roadway.

Table 71 compares the speeds at which skidding or rollover would occur for passenger cars and trucks traversing minimum radius curves designed in accordance with current AASHTO criteria. The table shows that on a dry pavement a passenger car will skid at a lower speed than it rolls over, and a truck with rollover threshold of 0.30 g will roll over at a lower speed than it skids. On a wet pavement, a passenger car will still skid at a lower speed than it rolls over. A truck, on the other hand, will skid before it rolls over at design speeds of 40 mi/h (64 km/h) or less under the assumed values for pavement friction on wet pavements. If a wet pavement has above minimum friction, however, the truck may still roll over at a lower speed than it skids. Finally, for curve design speeds over 40 mi/h (64 km/h), the truck will always roll over before it skids under the assumed design conditions.

Table 72 presents the results of an alternative sensitivity analysis to table 71 by showing the lateral accelerations that result from overdriving horizontal curves at speeds up to 20 mi/h (32 km/h) higher than the design speed. The table addresses curves designed to the AASHTO minimum radius for selected values of design speed and maximum superelevation rate. Curves designed with larger radii than the AASHTO minimum naturally produce lower lateral acceleration than those shown in table 72. The results shown in table 72 are very much in line with operational experience. At lower design speeds, overdriving of design speed by even a few miles per hour can produce side friction demands above the rollover thresholds of trucks. On the other hand, at higher design speeds, overdriving of the design speed by as much as 20 mi/h (32 km/h) does not produce enough lateral acceleration to produce a truck rollover.

4. Recommended Revisions to Design and Operational Criteria

Based on the sensitivity analysis reported above, there does not appear to be a need to modify existing criteria for determining the radius and superelevation of horizontal curves to accommodate trucks at particular design speeds. The margins of safety against skidding by trucks appear to be adequate even for trucks traveling at the design speed under wet pavement conditions. The margins of safety against rollover by trucks appear to be adequate even for trucks with extremely low rollover thresholds (0.30 g) traveling at

Table 71. Vehicle speed at impending skidding or rollover on horizontal curves.

Design speed (mi/h)	Maximum e	Maximum tolerable lateral acceleration	Minimum radius (ft)	Passenger car available cornering f	Passenger car speed (mi/h) @ impending skid (wet)	@ impending skid (dry)	@ rollover RT = 1.20 g	Truck speed (mi/h) @ impending skid (wet)	@ impending skid (dry)	@ rollover RT = 0.27 g	RT = 0.30 g	RT = 0.35 g	RT = 40 g
20	0.04	0.17	127	0.58	34.4	43.3	48.6	27.9	34.9	24.3	25.4	27.3	29.0
30	0.04	0.16	302	0.51	49.8	66.7	74.9	40.5	53.8	37.5	39.2	42.0	44.6
40	0.04	0.15	573	0.46	65.8	91.9	103.2	53.7	74.2	51.6	54.1	57.9	61.5
50	0.04	0.14	955	0.44	82.5	118.6	133.3	67.4	95.7	66.6	69.8	74.7	79.4
60	0.04	0.12	1,528	0.42	102.7	150.1	168.6	84.0	121.1	84.3	88.3	94.5	100.4
20	0.06	0.17	116	0.58	33.4	41.8	46.8	27.3	33.9	24.0	25.0	26.7	28.3
30	0.06	0.16	273	0.51	48.2	64.1	71.8	39.6	52.0	36.8	38.4	41.0	43.4
40	0.06	0.15	509	0.46	63.3	87.5	98.1	52.1	71.0	50.2	52.4	55.9	59.3
50	0.06	0.14	849	0.44	79.4	113.0	126.7	65.5	91.7	64.8	67.7	72.3	76.5
60	0.06	0.12	1,348	0.42	98.6	142.4	159.6	81.4	115.5	81.7	85.3	91.1	96.4
70	0.06	0.10	2,083	0.41	120.7	177.0	198.4	99.7	143.6	101.5	106.1	113.2	119.9
20	0.08	0.17	107	0.58	32.5	40.5	45.3	26.8	33.0	23.7	24.7	26.3	27.8
30	0.08	0.16	252	0.51	47.1	62.2	69.6	39.0	50.7	36.4	37.9	40.3	42.6
40	0.08	0.15	468	0.46	61.8	84.7	94.8	51.3	69.1	49.6	51.6	54.9	58.0
50	0.08	0.14	764	0.44	76.8	108.2	121.1	63.9	88.3	63.3	66.0	70.2	74.2
60	0.08	0.12	1,206	0.42	95.2	136.0	152.2	79.3	110.9	79.6	82.9	88.2	93.2
70	0.08	0.10	1,910	0.41	118.0	171.2	191.5	98.5	139.6	100.1	104.3	111.0	117.3
20	0.10	0.17	99	0.58	31.8	39.3	43.9	26.4	32.2	23.4	24.4	25.9	27.2
30	0.10	0.16	231	0.51	45.9	60.1	67.1	38.3	49.2	35.8	37.2	39.5	41.6
40	0.10	0.15	432	0.46	60.5	82.2	91.8	50.6	67.3	49.0	50.9	54.0	56.9
50	0.10	0.14	694	0.44	74.6	104.2	116.3	62.6	85.4	62.1	64.5	68.4	72.1
60	0.10	0.12	1,091	0.42	92.3	130.6	145.9	77.6	107.0	77.8	80.9	85.8	90.5
70	0.10	0.10	1,637	0.41	111.5	160.0	178.7	93.8	131.1	95.3	99.1	105.1	110.8

Note: 1 mi = 1.61 km
1 ft = 0.305

Table 72. Lateral acceleration developed by overdriving design speed for horizontal curves designed to AASHTO minimum radii.

Design speed (mi/h)	Maximum super-elevation (e_{max})	Maximum tolerable lateral acceleration	Minimum radius of curvature (ft)	Side friction demand for overdriving design speed of curve by: 0 mi/h	5 mi/h	10 mi/h	15 mi/h	20 mi/h
20	0.04	0.17	127	0.17	0.29	0.43	0.60	0.80
30	0.04	0.16	300	0.16	0.23	0.32	0.41	0.52
40	0.04	0.15	561	0.15	0.20	0.26	0.32	0.39
50	0.04	0.14	926	0.14	0.18	0.22	0.26	0.31
60	0.04	0.12	1,500	0.12	0.15	0.18	0.21	0.24
20	0.06	0.17	116	0.17	0.30	0.46	0.64	0.86
30	0.06	0.16	273	0.16	0.24	0.33	0.43	0.55
40	0.06	0.15	508	0.15	0.21	0.27	0.34	0.41
50	0.06	0.14	833	0.14	0.18	0.23	0.28	0.33
60	0.06	0.12	1,333	0.12	0.15	0.18	0.22	0.26
65	0.06	0.11	1,657	0.11	0.14	0.17	0.20	0.23
70	0.06	0.10	2,042	0.10	0.12	0.15	0.18	0.20
20	0.08	0.17	107	0.17	0.31	0.48	0.69	0.92
30	0.08	0.16	250	0.16	0.25	0.35	0.46	0.59
40	0.08	0.15	464	0.15	0.21	0.28	0.35	0.44
50	0.08	0.14	758	0.14	0.19	0.24	0.29	0.35
60	0.08	0.12	1,200	0.12	0.15	0.19	0.23	0.28
65	0.08	0.11	1,482	0.11	0.14	0.17	0.21	0.24
70	0.08	0.10	1,815	0.10	0.13	0.16	0.19	0.22
20	0.10	0.17	99	0.17	0.32	0.51	0.73	0.98
30	0.10	0.16	231	0.16	0.25	0.36	0.49	0.62
40	0.10	0.15	427	0.15	0.22	0.29	0.37	0.46
50	0.10	0.14	694	0.14	0.19	0.25	0.31	0.37
60	0.10	0.12	1,091	0.12	0.16	0.20	0.24	0.29
65	0.10	0.11	1,341	0.11	0.14	0.18	0.22	0.26
70	0.10	0.10	1,633	0.10	0.13	0.16	0.19	0.23

Note: 1 mi = 1.61 km
1 ft = 0.305 m

the design speed. Furthermore, a computer simulation study found that variations in the methods for providing superelevation transitions have only very small effects on the likelihood of skidding or rollover by trucks.

Although the design criteria themselves do not need to be changed, increased emphasis is needed on the selection of design speeds for particular curves, particularly on freeway ramps. It is evident from table 71 that trucks with extremely low rollover thresholds (0.30 g) can roll over on some curves when traveling as little as 5 mi/h (8 km/h) over the design speed. In most cases, a truck with a very low rollover threshold would not roll over unless it was traveling at least 10 mi/h (16 km/h) above the design speed.

Unfortunately, many freeway ramps have unrealistically low design speeds in comparison to the design speed of the mainline roadway. Table 73, taken from table X-1 of the Green Book, shows the existing criteria for selecting the design speed of a ramp as related to the highway design speed. If the lower range values for ramp design speed are used (as they often are), a 35-mi/h (56 km/h) off-ramp can be located on a 70-mi/h (113 km/h) highway. Speed differences as large as 35 mi/h (56 km/h) between the highway and the ramp are undesirable and can lead to trucks traveling fast enough to roll over on the ramp. Therefore, it is recommended that the lower range values in table 73 not be used on roadways that carry substantial volumes of heavy trucks. On existing curves where the lower range of design speeds have already been used, traffic control devices to inform truck drivers of the need to slow down are particularly important. Further research on increasing the effectiveness of such devices may be needed.

5. Summary

The evaluation of existing design criteria for selecting the radius and superelevation of horizontal curves at particular design speeds found that these criteria are adequate to accommodate trucks. In particular, current methods for superelevation transition between tangents and curves are adequate for trucks. Spiral transitions produce slightly less lateral acceleration than the 2/3-1/3 rule, but the difference is too small to suggest that increased use of spirals is likely to reduce substantial numbers of rollover accidents. However, it was also found that more emphasis needs to be placed on the selection of realistic design speeds for curves to minimize the likelihood of trucks traveling faster than the design speed. Selection of design speeds for freeway ramps that are consistent with the mainline highway design speed is particularly important. For this reason, the lower range values of ramp design speeds in table X-1 of the AASHTO Green Book should not be used on roadways that carry substantial volumes of truck traffic. Further research on methods to improve the effectiveness of traffic control devices on truck speeds may also be needed.

Table 73. Guidelines for ramp design speed as related to highway design speed.[1]

Highway Design Speed (mph)	30	40	50	60	65	70
Ramp Design Speed (mph)						
Upper Range (85%)	25	35	45	50	55	60
Middle Range (70%)	20	30	35	45	45	50
Lower Range (50%)	15	20	25	30	30	35
Corresponding Minimum Radius (ft)	SEE TABLE III-6					

Note: 1 mi = 1.61 km

L. Pavement Widening on Horizontal Curves

1. Current Highway Design and Operational Criteria

The AASHTO Green Book presents the current criteria for pavement widening on horizontal curves to accommodate offtracking of trucks. Offtracking is the phenomenon, common to all vehicles although much more pronounced with large trucks, in which the rear wheels do not track precisely behind the front wheels when the vehicle negotiates a horizontal curve.

The AASHTO criteria call for widening of curves according to tabulated criteria that depend on the pavement width on the tangent, the design speed, and the degree of curve. This table is reproduced here as table 74. It, in turn, is based on the formulas and definitions given in the Green Book as figure III-24. As noted with the table, the AASHTO policy is to disregard widening values less than 2 ft (0.6 m) and to add a certain amount to the tabulated values if the trucks using the facility are commonly "semitrailers" Moreover, the table applies only to two-lane roads (one- or two-way); the values given in the table are to be adjusted upward for three- or four-lane roads.

The AASHTO policy also details how the widening should be accomplished. In other words, it notes whether the added width should be on the inside or outside of the curve, how it should be transitioned, and how the center line should be adjusted.

Table 74. AASHTO criteria for pavement widening on horizontal curves.[1]

Degree of Curve	Widening (ft) for Two-Lane Pavements on Curves for Width of Pavement on Tangent of: 24 ft — Design Speed (mph) 30	40	50	60	70	22 ft — Design Speed (mph) 30	40	50	60	70	20 ft — Design Speed (mph) 30	40	50	60
1	0.0	0.0	0.0	0.0	0.0	0.5	0.5	0.5	1.0	1.0	1.5	1.5	1.5	2.0
2	0.0	0.0	0.0	0.5	0.5	1.0	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.5
3	0.0	0.0	0.5	0.5	1.0	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.5	2.5
4	0.0	0.5	0.5	1.0	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.5	2.5	3.0
5	0.5	0.5	1.0	1.0		1.5	1.5	2.0	2.0		2.5	2.5	3.0	3.0
6	0.5	1.0	1.0	1.5	–	1.5	2.0	2.0	2.5		2.5	3.0	3.0	3.5
7	0.5	1.0	1.5	–		1.5	2.0	2.5			2.5	3.0	3.5	
8	1.0	1.0	1.5			2.0	2.0	2.5			3.0	3.0	3.5	
9	1.0	1.5	2.0			2.0	2.5	3.0			3.0	3.5	4.0	
10-11	1.0	1.5				2.0	2.5				3.0	3.5		
12-14.5	1.5	2.0				2.5	3.0				3.5	4.0		
15-18	2.0					3.0					4.0			
19-21	2.5					3.5					4.5			
22-25	3.0					4.0					5.0			
26-26.5	3.5					4.5					5.5			

NOTES: Values less than 2.0 may be disregarded.

3-lane pavements: multiply above values by 1.5.

4-lane pavements: multiply above values by 2.

Where semitrailers are significant, increase tabular values of widening by 0.5 for curves of 10° to 16°, and by 1.0 for curves 17° and sharper.

1 ft = 0.305 m
1 mi = 1.61 km

2. Critique of Current Highway Design and Operational Criteria

The introductory portion of the AASHTO Green Book policy on pavement widening on curves illustrates that the authors had a good appreciation of the major factors that influence offtracking. These factors have been known for many years and were also quoted in the earlier AASHO Blue Book.[51] In recent years, and especially with the advent of the larger 1982 STAA vehicles, these factors have been studied in more depth, and their importance is becoming more apparent.

Unfortunately, the present design criteria are flawed in two major respects: (1) they have not changed since the 1965 Blue Book, even though the vehicles they intended to deal with have changed considerably; and (2) the criteria do not, in fact, correctly incorporate many of the factors that are stated to be important. These flaws are discussed in more detail below. They all deal with some aspect of offtracking, as defined above.

The policy also incorporates two other factors, front overhang and an extra width allowance, to account for the difficulty of maneuvering on a curve. These factors are not believed to be affected by the introduction of the 1982 STAA vehicles.

a. Hiqh-Speed Versus Low-Speed Offtracking

The AASHTO policy notes that there are two distinct types of offtracking. Low-speed offtracking is a purely geometrical phenomenon, wherein the rear axle(s) tracks toward the inside of a horizontal curve, relative to the front axle. Considerable research has been done to document the extent of this phenomenon, as a function of truck and roadway geometrics, and it is quite well understood. High-speed offtracking, on the other hand, is a dynamic phenomenon. It is caused by the tendency of the rear of the vehicle to move outward due to the lateral acceleration of the vehicle as it negotiates a horizontal curve at higher speeds. It is less well understood, and is a function of not only the truck and roadway geometrics, but also the vehicle speed and the suspension and tire characteristics of the vehicle. The current AASHTO criteria are based only on (an estimate of) the low-speed offtracking; no consideration of high-speed offtracking is included.

b. Superelevation

No consideration of superelevation is included in the policy. It is known (at least, based on observations) that low-speed offtracking is amplified with superelevation. However, this phenomenon has not been quantified prior to the present study.

c. Design Vehicle

Although the AASHTO policy discusses the importance of wheelbase in calculating offtracking, it then uses a single-unit truck as the primary design vehicle. It does footnote a correction for larger trucks, to be used if such trucks are common. The larger design vehicles mentioned here, as well as other places in the Green Book, are the WB-40 and WB-50 tractor-semitrailers.

The WB-40 is a light duty combination vehicle with a short, cab-over-engine tractor with a 35-ft (10.7-m) trailer, also commonly referred to as a 2-S2 configuration. The WB-50 is a conventional tractor with a 37-ft (11.3-m) trailer, a 3-S2 configuration. These were the tractor-semitrailers in common use in the late 1950's, when the Blue Book was prepared. The design criteria have not been upgraded since. As shown in appendix C in volume II of this report, the impact of longer trailers on offtracking is great, and should be accounted for.

d. Offtracking Formula

The design table is based on calculations from formulas given in figure III-24 of the policy. The dominant element in the formulas is the vehicle track width, also commonly referred to as swept width. The swept width is said to be determined as:

$$U = u + R - \quad R^2 - L^2 \tag{69}$$

where: U = swept width (ft)

u = vehicle track width on a tangent (ft)

R = Radius of the curve (ft)

L = "wheelbase" (ft)

The quotes around the term "wheelbase" have been added for editorial purposes, because the definition is not really correct. The Green Book states that the wheelbases of the design vehicles are as follows:

WB-40: Wheelbase = 13+27 = 40
WB-50: Wheelbase = 20+30 = 50

The unknowing reader might be tempted to substitute these values into equation (69) to calculate the vehicle track width. In fact, equation (69) is valid only for single-unit vehicles. For articulated vehicles, an alternative version, such as the Western Highway Institute model, must be used:[77]

$$U = u + R - R^2 - \Sigma (L_i^2) \tag{70}$$

where the L_i are the distances between consecutive axles (or sets of tandem axles) and articulation points. For example, for a tractor-semitrailer there are three values:

L_1, the distance from the front axle to the tractor drive axle(s),

L_2, the distance from the drive axle(s) to the fifth wheel pivot, and

L_3, the distance from the fifth wheel pivot to the rear axle(s).

In the summation process, the second term is subtracted, rather than added, because the fifth wheel pivot is generally in front of, rather than behind, the tractor drive axle(s).

The difference in the values obtained using the two equations can be substantial, and is accentuated as the number of articulation points increases, as it does with doubles and triples. The more simplified formula overstates the amount of offtracking. It is our belief that the numerical values in the Green Book were obtained using the correct formula [(equation (70)] however.

e. Length of Curve

The offtracking formulas given above are fairly simple, in large measure because they represent a special case of offtracking--which we might call "fully developed" offtracking. As a vehicle leaves a tangent section and begins to traverse a curve, the amount of offtracking changes with distance. It is initially zero, but it increases as the truck proceeds around the curve. Finally it approaches, asymptotically, the "fully developed" amount of offtracking. In effect, the track of the rear axle spirals outward, from being on the same track as the front axle on the tangent, to its final distance reflected by U in equation (70).

The determination of the development of offtracking with distance around a horizontal curve is not easy. It is typically accomplished with a scale model, a template, or a computer program. The answer depends on the radius of the curve and the geometrics of the vehicle.

3. <u>Sensitivity Analyses</u>

A sensitivity analysis was conducted to determine the pavement width requirements to accommodate trucks on horizontal curves. Based on the critique of the existing AASHTO criteria given above, this analysis uses a slightly different approach to pavement widening criteria. Those criteria are based solely on traditional low-speed offtracking and do not consider either the speed-dependent ("high-speed") component or the superelevation component of offtracking and address a design vehicle that is too small for current conditions. The sensitivity analysis reported here determined the lane width required to accommodate trucks under various offtracking scenarios; the required pavement widening is not specified directly, but could be computed as the difference between the lane width required on the curve and the actual lane width on the tangent.

Appendix C in volume II of this report presents a new model for computing offtracking on horizontal curves. This model not only computes the traditional low-speed component of offtracking, but also incorporates the effects of vehicle speed and pavement superelevation on offtracking. The model is equivalent to the Western Highway Institute offtracking model, in that it computes the fully developed offtracking, rather than the actual offtracking on shorter curves, which may be less than the fully developed offtracking, and can be computed with computer models, such as the Caltrans offtracking model, that plot the turning paths of trucks making specific maneuvers.[72,77]

At low speed, the rear axle of any vehicle making a turn will follow a path inside of the front axle. The speed-dependent component of offtracking increases with the square of the vehicle speed and acts in the opposite direction to the low-speed component; i.e., as speed increases, the rear axle of the vehicle moves back toward the path of the front axle and, at very high speeds, it may actually track outside of the front axle. There is also a superelevation effect that is independent of speed and tends to make the rear axle track further to the inside. Following the sign convention used in the appendix C model, offtracking toward the inside of the curve will be referred to as "negative offtracking," and offtracking toward the outside of the curve will be referred to as a positive offtracking.

The lane width requirements on horizontal curves were based on the STAA 48-ft (14.6-m) single-semitrailer truck (see table 3) as the design vehicle. The requirements indicate the minimum lane widths required for maneuvers by that truck, such that the roadway is wide enough to accommodate its swept path width plus a 1 ft (0.3 m) clearance on both sides. The swept path width was computed as the fully developed offtracking from the model in appendix C plus 7.58 ft (2.31 m), which represents half of the width of the tractor steering axle plus half of the width of the rear trailer axle.

The offtracking model incorporates a number of specific truck design parameters that influence offtracking to some extent. Three offtracking scenarios were considered, which used various combinations of these parameters. These are: (1) typical values of the parameters for a loaded truck (specified in tables 24 and 25 of appendix C in volume II; (2) the combination of parameters that produces the largest negative offtracking; and (3) the combination of parameters that produces the largest positive offtracking. The largest negative offtracking occurs for an empty truck; typical values of axle load and center of gravity heights for empty trucks are also given in appendix C. The largest positive offtracking occurs for trucks with relatively low values of cornering coefficient and composite roll stiffness, which are also given in appendix C.

Table 75 presents minimum lane widths for each of these three scenarios for selected combinations of curve radius and superelevation to serve as a basis for determining the need for pavement widening or horizontal curves. These combinations are selected to represent the minimum radius curve for each value of design speed and maximum superelevation (e_{max}) and selected larger radii up to 2,000 ft (610 m). Offtracking is usually not a major consideration on curves with radii above 2,000 ft (610 m). The minimum lane width for each scenario is the largest lane width required at any speed less than or equal to the design speed. Table 75 is not meant to suggest that any particular lane width is acceptable or unacceptable for a particular roadway class. Other AASHTO criteria address that issue (see section II-K). Instead, if a tangent roadway section has 11-ft (3.4-m) lanes, and table 75 indicates that a particular horizontal curve requires 13-ft (4.0-m) lanes, then it is recommended that the pavement be widened 2 ft (0.6 m) per lane on that curve.

The design value of minimum lane width presented in the table is the largest of the values for the three scenarios. The design values presented in the table range from 10.5 to 14 ft (3.2 to 4.3 m). The minimum lane width of 14 ft (4.3 m) occurs only for the largest positive offtracking by a truck traveling at the design speed on a curve with the AASHTO minimum radius for a superelevation of 0.10.

4. Recommended Revisions to Highway Design and Operational Criteria

It is recommended that existing criteria for pavement widening on horizontal curves be changed to use a larger design vehicle and a more complete offtracking model. The recommended criteria, presented in table 75, would provide enough pavement width for offtracking by the recommended STAA

Table 75. Minimum lane width required to accommodate truck offtracking on horizontal curves.

				Minimum lane width (ft)[a]			
Design speed (mi/h)	Maximum super elevation	Radius (ft)	Actual super elevation	Typical truck characteristics	Truck characteristics for largest negative offtracking	Truck characteristics for largest positive offtracking	Design values
40	0.04	573	0.040	12.0	11.5	12.0	12.0
		600	0.040	11.5	11.5	12.0	12.0
		800	0.037	11.5	11.0	11.5	11.5
		1,000	0.035	11.0	11.0	11.5	11.5
		1,500	0.030	10.5	10.5	11.0	11.0
		2,000	0.027	10.5	10.5	10.5	10.5
50	0.04	955	0.040	11.0	11.0	11.5	11.5
		1,000	0.040	11.0	11.0	11.5	11.5
		1,500	0.037	10.5	10.5	11.0	11.0
		2,000	0.033	10.5	10.5	11.0	11.0
60	0.04	1,528	0.040	10.5	10.5	11.0	11.0
		2,000	0.039	10.5	10.5	11.0	11.0
40	0.06	509	0.060	12.0	12.0	13.0	13.0
		600	0.059	12.0	11.5	12.5	12.5
		800	0.056	11.5	11.0	12.0	12.0
		1,000	0.050	11.0	11.0	11.5	11.5
		1,500	0.041	10.5	10.5	11.0	11.0
		2,000	0.034	10.5	10.5	10.5	10.5
50	0.06	849	0.060	11.5	11.0	12.0	12.0
		1,000	0.059	11.0	11.0	12.0	12.0
		1,500	0.052	10.5	10.5	11.5	11.5
		2,000	0.045	10.5	10.5	11.0	11.0
60	0.06	1,348	0.060	11.0	10.5	11.5	11.5
		1,500	0.060	11.0	10.5	11.5	11.5
		2,000	0.055	10.5	10.5	11.0	11.0
70	0.06	2,083	0.060	10.5	10.5	11.5	11.5
40	0.08	468	0.080	12.5	12.0	13.5	13.5
		600	0.078	12.0	11.5	13.0	13.0
		800	0.071	11.5	11.0	12.0	12.0
		1,000	0.062	11.0	11.0	11.5	11.5
		1,500	0.047	10.5	10.5	11.0	11.0
		2,000	0.038	10.5	10.5	10.5	10.5

Table 75. Minimum lane width required to accommodate truck offtracking on horizontal curves. (continued)

Design speed (mi/h)	Maximum super elevation	Radius (ft)	Actual super elevation	Minimum lane width (ft)[a]: Typical truck characteristics	Truck characteristics for largest negative offtracking	Truck characteristics for largest positive offtracking	Design values
50	0.08	764	0.080	11.5	11.0	12.5	12.5
		800	0.080	11.5	11.0	12.5	12.5
		1,000	0.077	11.0	11.0	12.0	12.0
		1,500	0.063	11.0	10.5	11.5	11.5
		2,000	0.053	10.5	10.5	11.0	11.0
60	0.08	1,206	0.080	11.0	11.0	12.0	12.0
		1,500	0.078	11.0	10.5	12.0	12.0
		2,000	0.068	10.5	10.5	11.5	11.5
70	0.08	1,910	0.080	11.0	10.5	11.5	11.5
		2,000	0.080	11.0	10.5	11.5	11.5
40	0.10	432	0.100	12.5	12.0	14.0	14.0
		600	0.094	12.0	11.5	13.0	13.0
		800	0.084	11.5	11.0	12.5	12.5
		1,000	0.070	11.0	11.0	11.5	11.5
		1,500	0.051	10.5	10.5	11.0	11.0
		2,000	0.040	10.5	10.5	10.5	10.5
50	0.10	694	0.100	12.0	11.5	13.0	13.0
		800	0.100	11.5	11.0	13.0	13.0
		1,000	0.092	11.5	11.0	12.5	12.5
		1,500	0.072	11.0	10.5	11.5	11.5
		2,000	0.057	10.5	10.5	11.0	11.0
60	0.10	1,091	0.100	11.5	11.0	12.5	12.5
		1,500	0.093	11.0	10.5	12.0	12.0
		2,000	0.077	10.5	10.5	11.5	11.5
70	0.10	1,637	0.100	11.0	10.5	12.0	12.0
		2,000	0.096	11.0	10.5	12.0	12.0

[a] Based on STAA 48-ft (14.6-m) semitrailer with conventional tractor as design vehicle.
Note: 1 mi = 1.61 km; 1 ft = 0.305 m.

48-ft (14.6-m) design vehicle at speeds up to the design speed for scenarios including typical truck characteristics, and the truck characteristics that provide the largest negative and the largest positive offtracking. The required amount of pavement widening per lane on a horizontal curve is the difference between the design value in table 75 and the actual lane width on the tangent section.

M. Cross-Slope Breaks

1. Current Highway Design and Operational Criteria

The following represents a brief summary of the AASHTO Green Book criteria for cross-slope rates:

- On tangent or long-radius curved alignment with normal crown and turf shoulders, the maximum shoulder slope rates result in algebraic differences of 6 to 7 percent between the pavement and the shoulder.

- For desirable operation, all or part of the shoulder on the outside of a horizontal curve should be sloped upward at about the same rate or at a lesser rate than the superelevated pavement.

- The cross-slope break at the edge of the paved surface is limited to a maximum of approximately 8 percent.

- To alleviate severe cross-slope breaks, the use of a continuously rounded shoulder cross section may be used on the outside of superelevated pavements.

2. Critique of Current Highway Design and Operational Criteria

a. Cross-Slope Breaks

A 1982 FHWA study investigated the operational effects of cross-slope breaks on highway curves.[91] Using the Highway-Vehicle-Object Simulation Model (HVOSM), vehicle traversals were simulated for various combinations of pavement and shoulder slopes for a range of horizontal curvature. The objective criterion was to limit lateral acceleration to a level that was stable at the tire-pavement interface and tolerable to the driver. A 1971 Dodge Coronet was the passenger car used in the simulations.

The study results indicated that a four-wheel traversal and entry to a cross-slope break produce a more extreme response than a two-wheel traversal. The dynamic effects were found to be most sensitive to shoulder cross-slope and to exceed reasonable driver discomfort levels for the design conditions that reduce the conditions associated with higher cross-slope breaks. It was determined that relatively large negative slopes are tolerable on very narrow shoulders. As shoulder width increases, permissible shoulder slopes should decrease to maintain the established maximum driver discomfort level. Specifically, the study found that maximum driver discomfort occurred when all four wheels were on the shoulder, not when the vehicle crosses the break.

The FHWA study identified two unanswered questions regarding the sensitivity of trucks to cross-slope break traversals:[92]

1. Do professional (truck) drivers exhibit higher tolerable levels of driver discomfort?

2. Do shoulder traversals by trucks occur often enough to justify the truck as the "design" vehicle for cross-slope break recommendations?

No further data were found in the literature to shed any additional light on these issues.

b. Centerline Crowns

In another portion of the same FHWA study that was discussed above, the dynamic effects of centerline crowns on expected vehicle maneuvers were evaluated for the purpose of recommending maximum centerline crown designs as a function of vehicle type and design speed.[93] The controlling operational maneuver was the passing situation. Research was limited to tangent roadway sections. Vehicle types considered included: compact and mid-size passenger cars, loaded and empty tractor-trailer truck combinations, and single-unit trucks.

The pertinent truck-related findings include:

- A loaded or empty tractor-trailer truck generates lower tire friction demand than automobiles on 2 percent cross slopes.

- Driver discomfort levels and vehicle roll angle are also less for trucks than automobiles on 2 percent cross slopes at high speed (approximately 75 mi/h or 121 km/h).

- An empty tractor-trailer produces similar tire friction demands (approximately 0.30 g), but has significantly lower driver discomfort and roll angle values.

The implication of the findings is that cross-slope design should be kept to a minimum on high-speed highways. The primary reason is that the simulation of nominally critical passing behavior produced vehicle dynamic responses on the order of 0.28 to 0.34 g for cross slopes of 2 percent for all vehicle types.

3. Sensitivity Analyses

No sensitivity analyses were performed for this issue.

4. Summary

There are no data available to determine whether the current AASHTO criterion for the maximum cross-slope break at the edge of the traveled way on a horizontal curve is adequate for trucks. Research on cross-slope breaks at the centerline crown of a highway indicates that the roadway cross-slope should be kept to a minimum to maintain safe truck operations in passing maneuvers.

N. Roadside Slopes

1. Current Highway Design and Operational Criteria

The AASHTO Green Book states that "sideslopes should be designed to ensure the stability of the roadway and to provide a reasonable opportunity for recovery for an out-of-control vehicle." The use of roadside slopes as steep as 3:1 is implicitly permitted under AASHTO criteria, because roadside barriers are not warranted.[94] When slopes steeper than 3:1 are used, AASHTO recommends the consideration of a roadside barrier. On freeways and other arterials, a sideslope of 6:1 is recommended to provide a good chance of recovery. Depending on the hazard at the toe of slope, steeper slopes up to about 3:1 are considered traversable on lower functional classes of roadways. Roadside slopes should be free of obstacles within a clear zone up to 30 ft (9 m) from the edge of the traveled way. Wider clear zones are required on horizontal curves.[94]

2. Critique of Highway Design and Operational Criteria

Although the design vehicle used to develop the AASHTO criteria for roadside slopes is not stated, the largest vehicle used in most full-scale crash tests has been an automobile weighing approximately 4,500 lb (2,050 kg) or less. Some roadside barriers have been designed for containing and/or redirecting large trucks and buses.[94] However, there have been no full-scale tests of trucks traversing various side slope combinations. Thus, the current criteria for roadside slopes do not consider trucks.

It is known that trucks are prone to rollovers since they have a high center of gravity compared to passenger cars. Once the truck's lateral acceleration exceeds its rollover threshold, a rollover may be initiated. Any adverse cross slope increases the opportunity for a truck to rollover. As such, once a truck leaves the traveled way (and traversable shoulder) and enters the roadside area, a rollover can result.

There is no information available regarding the critical combinations of traveled way cross-slope, shoulder cross-slope, and roadside slope combinations that either contribute to the roll propensity of trucks or offer them an opportunity to recover from a roadside traversal. If small angle departures from the traveled way are considered, the influence of various cross slope combinations could be identified that decrease the probability of a rollover event. Likewise, critical or limiting values could be determined that may warrant roadside barriers where truck traffic volumes are significant, even though the roadside configuration is considered traversable by a passenger car.

3. Summary

The issue of roadside slope design for trucks has not been addressed in past research. Therefore, no conclusions can be drawn concerning the adequacy of current roadside slope design criteria for trucks.

O. Vehicle Change Interval

1. Current Highway Design and Operational Criteria

The vehicle change interval at a signalized intersection is the yellow signal phase provided at the end of the green phase to warn drivers of the impending change in right-of-way assignment. Section 4B-15 of the MUTCD specifies that the yellow vehicle change interval should range in length from approximately 3 to 6 s. The MUTCD states that the longer signalized intersections are generally appropriate for higher approach speeds. The MUTCD does not provide any other guidance on the selection of the length of the vehicle change interval. However, the MUTCD states that the yellow vehicle clearance interval may be followed by a short all-way red clearance interval of sufficient duration to permit the intersection to clear before traffic is released.

2. Critique of Current Highway Design and Operational Criteria

The FHWA Traffic Control Devices Handbook (TCDH) provides a more complete examination of the issues involved in selection of a vehicle change interval.[95] First, the TCDH refers to this interval as a "phase change interval," which is more descriptive of its function. Second, the TCDH suggests that, because excessively long yellow intervals may encourage driver disrespect, a maximum yellow interval of about 5 s should be used. If a longer phase change interval is needed, then the additional time should be provided with an all-red interval. Finally, the TCDH presents several alternative methods for determining the length of the phase change interval (yellow plus all-red time).

The TCDH states that some authorities believe that the timing of a phase change interval should enable a vehicle traveling in the direction in which the yellow signal is displayed to clear the intersection before the onset of the green phase for conflicting movements. In this case, the length of the phase change interval is determined as the sum of a perception-reaction period, a deceleration period, and an intersection-clearing period, as follows:

$$Y + AR = t_{pr} + \frac{V}{2d + 0.644\ g} + \frac{W + L}{V} \qquad (71)$$

where: $Y + AR$ = phase change interval (yellow plus all-red time) (s)

t_{pr} = driver perception and brake reaction time (s)

V = approach speed (ft/s)

d = deceleration rate (ft/s^2)

g = percent grade (+ for upgrade, - for downgrade)

W = width of intersection (ft)

L = length of vehicle (ft)

Equation (71) at first appears incorrect to casual readers because it involves both a deceleration rate term (which implies that the vehicle will stop at the signal) and a clearance term (which implies that the vehicle will proceed through the intersection). However, the derivation of equation (71) is correct. The first two terms represent the time required for a vehicle traveling at the prevailing speed (V) to reach the stop line from the closest point to the intersection at which it could stop within a specified deceleration rate (d). It is implicity assumed in this relationship that the driver makes the correct decision between stopping and continuing through the intersection based on his speed and location at the moment when the signal turns yellow.

Equation (71) is a very conservative policy in that approaching vehicles are provided an opportunity to clear the entire intersection before the green signal is displayed to conflicting traffic, even though the conflicting traffic is usually stopped and requires some additional time to accelerate from a stop and reach a point of conflict with the clearing vehicle. Field observations in a 1984 FHWA study found a mean starting time of 1.8 s for cross traffic, although a small fraction of vehicles (0.8 percent) started before their signal turned green.[96]

The TCDH points out that some jurisdictions follow a policy of allowing the onset of green for a conflicting approach after vehicles have partially cleared the intersection (i.e., after the rear of the vehicle has cleared the centerline of the conflicting approach). This policy should consider the geometrics of each specific intersection to which it is applied, but it can be approximated by replacing the term W by W/2 in equation (71).

The TCDH also points out that some jurisdictions time the phase change interval to allow the onset of the green phase for conflicting movements without the intersection having been cleared. The following equation is used in such cases:

$$Y + AR = t_{pr} + \frac{V}{2d + 0.644\,g} \qquad (72)$$

The use of this formulation for the vehicle change interval can create a dilemma zone within which some drivers can neither stop safely or clear the intersection safety.

The use of equation (71) is certainly the most prudent course for establishing the length of the vehicle change interval and it will be used for sensitivity analyses of the vehicle change interval in this report. However, there is no general agreement on the most appropriate formulation of the vehicle change interval. In fact, as some observers have noted, the Uniform Vehicle Code and the laws of most States allow drivers to enter an intersection at any point during a yellow signal phase.[84,97] A 1978 FHWA survey found that the procedure used for determining the length of the change interval was statistically independent of the State law regarding the meaning of the yellow indication.[98]

The TCDH recommends the use of 1.0 s for perception-reaction time (t_{pr}), 10 ft/s² (3 m/s²) for deceleration rate (d), and 20 ft (6.1 m) for vehicle length (L) in equation (71). The recommended values for d and L are appropriate for passenger cars, but not for trucks.

Recent field studies by found that the average deceleration rate (d) used by drivers in stopping for yellow signals ranged from 8.0 to 10.5 ft/s² (2.4 to 3.2 m/s²), as a function of approach speed. Observed perception-reaction times (t_{pr}) varied from 1.0 to 1.5 s, also as a function of approach speed. Table 76 illustrates the values of t_{pr} and d recommended for operational use by the 1984 FHWA study.[96] In contrast to the TCDH criteria, The FHWA study also suggested alternative methods for determining vehicle change interval lengths based on the observed distributions of the probability that a vehicle will be able to clear the intersection or stop for the signal. However, all of the FHWA study results are based solely on passenger cars and do not consider truck behavior on signal approaches.

3. Sensitivity analysis

A sensitivity analysis of the differences in vehicle change interval requirements for passenger cars and trucks was conducted. This sensitivity analysis compared the vehicle change interval requirements based on the TCDH criteria for passenger cars, based on the criteria for passenger cars from the 1984 FHWA study and based on estimated data for trucks. The criteria that were varied in this sensitivity analysis were:

- Perception-reaction time (t_{pr})
  - based on TCDH criteria (see table 76) for passenger cars
  - based on criteria from the 1984 FHWA study (see table 76 for passenger cars
  - based on use of criteria from the 1984 FHWA study as the estimated value for trucks

Table 76. Recommended passenger car performance criteria for determining vehicle change interval.

Approach speed (mi/h)	Traffic Control Devices Handbook[95] Perception-reaction time (t_{pr}) (s)	Deceleration rate (ft/s^2)	(g)	1984 FHWA study[96] Perception-reaction time (t_{pr}) (s)	Deceleration rate (ft/s^2)	(g)
25	1.0	10.0	0.31	1.5	8.0	0.25
30	1.0	10.0	0.31	1.4	8.5	0.26
35	1.0	10.0	0.31	1.3	9.0	0.28
40	1.0	10.0	0.31	1.2	9.5	0.30
45	1.0	10.0	0.31	1.1	10.0	0.31
50	1.0	10.0	0.31	1.0	10.5	0.33
55	1.0	10.0	0.31	1.0	10.5	0.33

- Deceleration rate (d)

  - based on TCDH criteria (see table 76) for passenger cars
  - based on criteria from the 1984 FHWA study (see table 76) for passenger cars
  - based on an estimated value of 5 ft/s² (1.5 m/s²) for trucks

- Percent grade (g)

  - 3 percent upgrade
  - level
  - 3 percent downgrade

- Length of vehicle (L)

  - 19 ft (6 m) for passenger cars
  - 75 ft (23 m) for trucks

- Width of intersection (W)

  - 40 ft (12 m) for moderate width intersection
  - 100 ft (31 m) for wide intersection

No data were available on the perception-reaction time requirements for braking by trucks. For analysis purposes, the perception-reaction times for trucks approaching yellow signals were assumed to be equal for the values for passenger cars observed by in the 1984 FHWA study.[96]

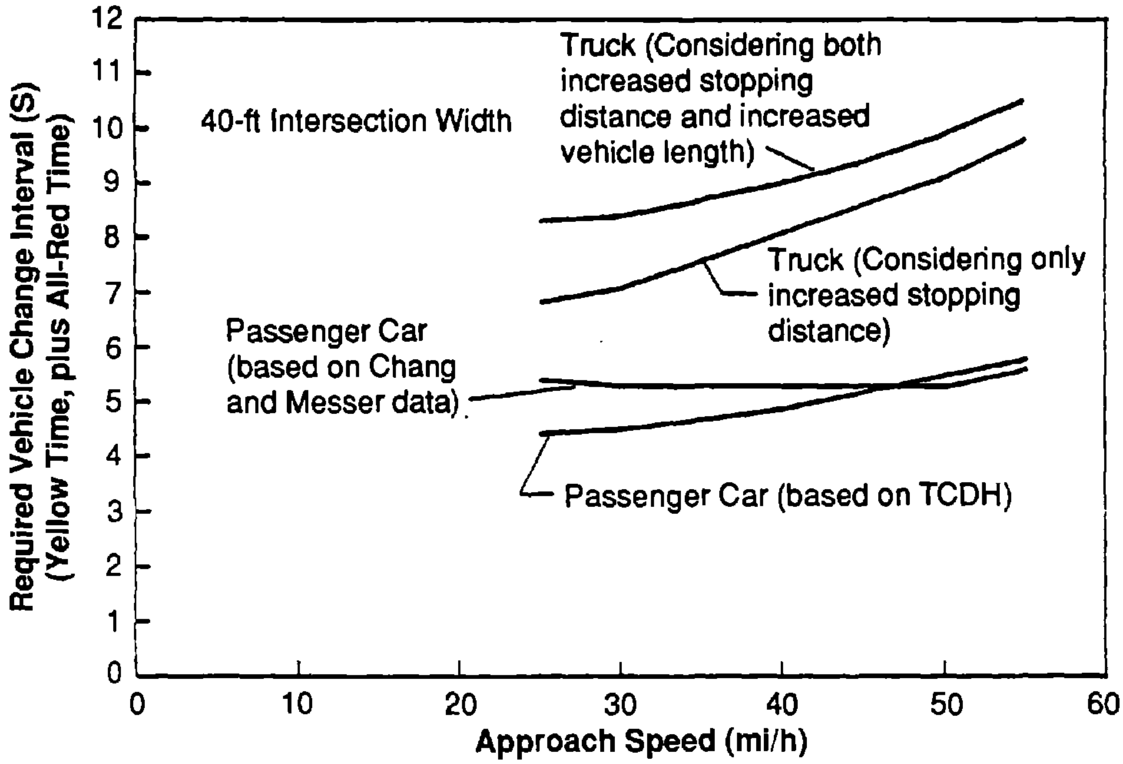
No data were available on the deceleration rates used by trucks approaching a yellow signal. The estimated rates for passenger cars (8 to 10.5 ft/s² or 2.4 to 3.2 m/s²) are within the capability of most trucks on a dry pavement, but may exceed the braking capabilities on a poor, wet pavement for trucks with inexperienced drivers. The deceleration rate for trucks in this analysis was assumed to be 5 ft/s² (1.5 m/s²), which is a comfortable rate on a dry pavement but may be a critical rate for some drivers on a poor, wet pavement.

Table 77 compares the length of the required vehicle change interval, based on equation (71), for the range of conditions discussed above. The vehicle change intervals range from 4.3 to 13.2 s depending on the criteria being evaluated. Any vehicle change interval requirements over 5.0 s would generally be met by a combination of yellow and all-red phases. Figure 46 summarizes the data shown in table 77. The figure also compares the required vehicle change interval for trucks based solely on their increased braking distances to the required vehicle change interval incorporating both their increased braking distances and their increased lengths.

Table 77. Sensitivity of vehicle change interval(s) to differences between passenger cars and trucks.[95,96]

Approach speed (mi/h)	40-ft intersection width: Vehicle change interval for passenger cars (TCDH)[95]	40-ft intersection width: Vehicle change interval for passenger cars (1984 FHWA study)[96]	40-ft intersection width: Estimated vehicle change interval for trucks	100-ft intersection width: Vehicle change interval for passenger cars (TCDH)[95]	100-ft intersection width: Vehicle change interval for passenger cars (1984 FHWA study[96]	100-ft intersection width: Estimated vehicle change interval for trucks
Grade: 3% Upgrade						
25	4.3	5.2	7.7	5.9	6.8	9.3
30	4.3	5.1	7.7	5.7	6.4	9.1
35	4.5	5.0	7.8	5.7	6.2	9.0
40	4.7	5.0	8.1	5.7	6.0	9.1
45	4.9	5.0	8.4	5.8	5.9	9.3
50	5.1	5.0	8.7	6.0	5.8	9.5
55	5.4	5.2	9.2	6.2	6.0	9.9
Grade: Level (%)						
25	4.4	5.4	8.3	6.1	7.0	9.9
30	4.5	5.3	8.4	5.9	6.7	9.8
35	4.7	5.3	8.7	5.9	6.5	9.8
40	4.9	5.3	9.0	6.0	6.3	10.0
45	5.2	5.3	9.4	6.1	6.2	10.4
50	5.5	5.3	9.9	6.3	6.1	10.7
55	5.8	5.6	10.5	6.5	6.3	11.2
Grade: 3% Downgrade						
25	4.6	5.7	9.2	6.3	7.4	10.8
30	4.8	5.7	9.5	6.1	7.0	10.8
35	5.0	5.6	9.9	6.2	6.8	11.1
40	5.3	5.6	10.4	6.3	6.7	11.5
45	5.5	5.6	11.0	6.5	6.6	11.9
50	5.9	5.7	11.7	6.7	6.5	12.5
55	6.2	6.0	12.4	6.9	6.7	13.2

Note: 1 mi = 1.61 km; 1 ft = 0.305 m.



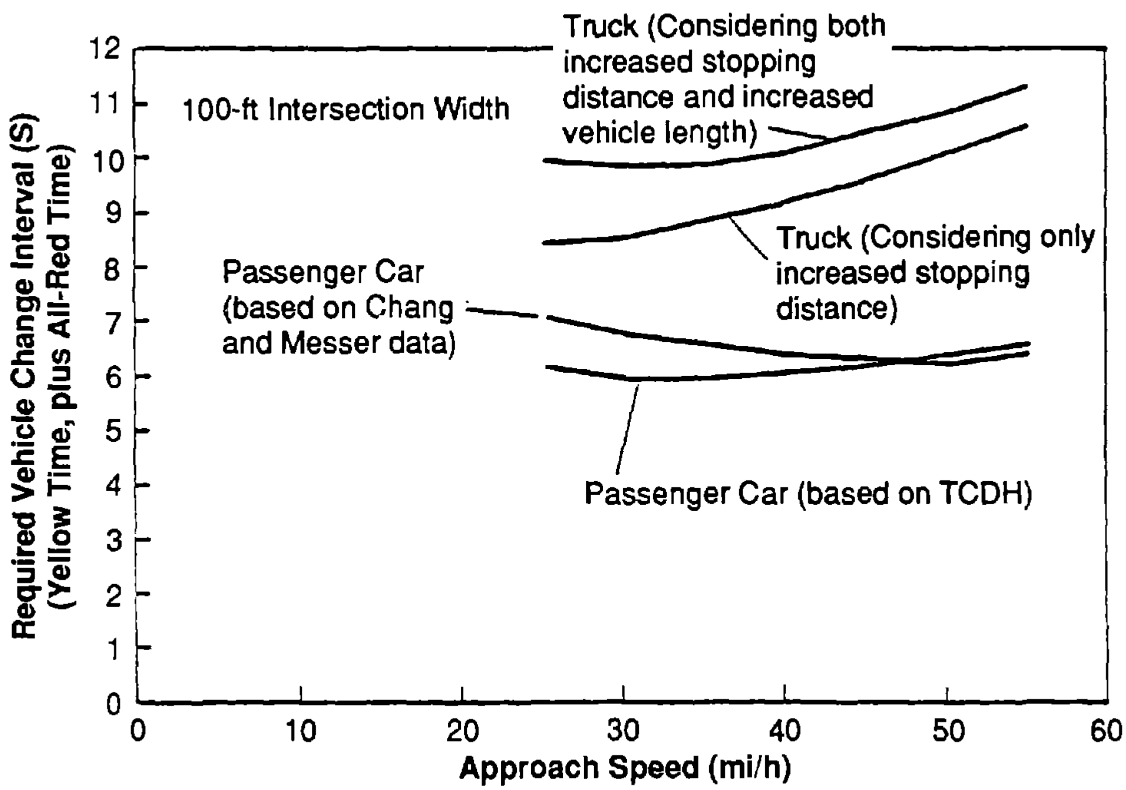


Figure 46. Required vehicle change intervals at signalized intersections for passenger cars and trucks.

For the 40-ft (12-m) intersection width, the required vehicle change intervals for trucks in table 77 are 50 to 110 percent higher than the passenger car requirements, based on the 1984 FHWA data.[95] At speeds below 40 mi/h (64 km/h), most of the added yellow time for trucks is due to their increased length. However, at higher speeds, the difference in deceleration rates between trucks and passenger cars also plays a major role in increasing the vehicle change interval requirements for trucks.

For the 100-ft (31-m) intersection width, the required vehicle change intervals for trucks in table 77 are 40 to 100 percent higher than the passenger car requirements, based on the 1984 FHWA data. Thus, the differences between passenger cars and trucks are slightly less critical at wider intersections. As at the narrower intersection, the deceleration rate begins to dominate the vehicle length in its contribution to vehicle change interval requirements at higher speeds.

The differences between passenger cars and trucks in vehicle change interval requirements are generally lowest on upgrades and greatest on downgrades.

4. Recommended Revisions to Highway Design and Operational Criteria

A sensitivity analysis shows that trucks require vehicle change intervals that are 40 to 110 percent longer than passenger cars, depending upon approach speed, approach grade, and intersection width. Longer vehicle change intervals could probably increase safety for trucks in some situations. However, a complete analysis of this issue should also consider the operational impact of the reduction in the duration of the green phases that would result from lengthening the clearance intervals. Not only would this increase operational delays at signals, but it might also create safety problems due to increased congestion. No change in existing criteria for vehicle change intervals is recommended based on the information currently available.

P. Sign Placement

This section reviews the suitability for trucks of the criteria for placement of signs.

1. Current Highway Design and Operational Criteria

Criteria for horizontal, vertical, and longitudinal placement of signs are presented in part II of the MUTCD.

a. Horizontal Placement

According to section 2A-24 of the MUTCD, roadside signs in rural areas are generally placed so that there is at least 6 ft (1.8 m) horizontal clearance from the outside edge of the shoulder to the nearest edge of the sign. If there is no shoulder, there should be at least 12 ft (3.7 m) horizontal clearance from the edge of the traveled way to the nearest edge of the sign. Longer clearances than 6 to 12 ft (1.8 to 3.7 m) are desirable on expressways and freeways, especially for large guide signs. A lesser clearance may be used in urban areas where necessary. Signs in urban areas should generally be at least 2 ft (0.6 m) from the edge of the traveled way, although a clearance of 1 ft (0.3 m) from the curb face is permissible where the sign position is restricted by the presence of a sidewalk or the location of existing poles.

b. Vertical Placement

According to section 2A-23 of the MUTCD, roadside signs in rural areas should be mounted at a height of at least 5 ft (1.5 m), measured from the bottom of the sign to the near edge of the pavement. In business, commercial, or residential districts where parking or pedestrian movements are likely or there are other obstructions to view, the clearance to the bottom of the sign should be at least 7 ft (2.1 m). The mounting height of a secondary sign, located beneath another sign, may be 1 ft (0.3 m) less than the minimum heights prescribed above.

On expressways and freeways, guide signs should be mounted with a minimum vertical clearance of 7 ft (2.1 m). When a secondary sign is mounted below another sign, the minimum vertical clearance to the major sign should be at least 8 ft (2.4 m).

The MUTCD specifies that overhead signs should be mounted to provide a minimum vertical clearance of at least 17 ft (5.2 m) over the entire width of the pavement and shoulders except where a lesser vertical clearance is used for the design of other overhead structures. This MUTCD criterion is consistent with the AASHTO Green Book which specifies a desirable vertical clearance of 16.5 ft (5.0 m) for overhead structures, with 14.5 ft (4.4 m) as a recommended minimum.

c. Longitudinal Placement

Table II-1 in MUTCD section 2C-3 presents criteria for advance placement of warning signs, shown here in table 78. These advance placement distances are intended to provide adequate time for drivers to perceive a potentially hazardous condition, identify the condition, decide what maneuver to make, and begin to perform that maneuver. The time required for this process is referred to as Perception-Intellection-Emotion-Volition (PIEV) time. Required PIEV time is generally thought to range from 3 to 10 s depending upon the nature of the potential hazard. Table 78 provides suggested minimum sign placement distances that are applicable to three specific conditions:

Condition A--a high driver judgment condition which requires the driver to use extra time in making and executing a decision because of a complex driving situation; i.e., lane changing, passing, or merging. Warning signs appropriate for this condition include MERGE, RIGHT LANE ENDS, etc. Condition A corresponds to the same situation types for which decision sight distance to a physical decision point such as a major fork is required (see discussion in section III-C) is needed.

Condition B--a condition in which the driver will likely be required to stop. Warning signs appropriate for this condition include CROSS ROAD, STOP AHEAD, SIGNAL AHEAD, PED-XING, etc.

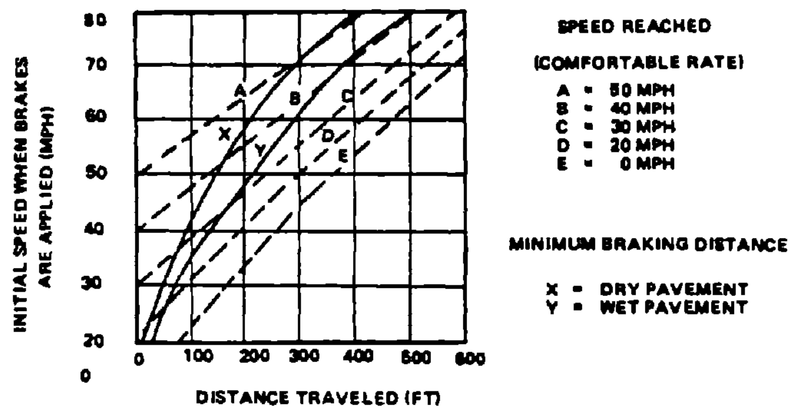
Condition C--a condition in which the driver will likely be required to decelerate to a specific speed. Warning signs appropriate for this condition include TURN, CURVE, DIVIDED ROAD, HILL, DIP, etc.

The values for condition A in table 78 provide 10 s travel time at the posted or 85th percentile speed. The values for Condition B in table 78 are based on the comfortable braking distances for passenger cars, shown in Line E of figure II-13 of the AASHTO Green Book, presented here as figure 47. The values for condition C in table 61 are based on the comfortable deceleration rates for passenger cars shown in lines A through D in figure 47. All of the values for conditions A, B, and C assume that the driver perception time begins at the point where the sign becomes legible, which is assumed to be 125 ft (38 m) in advance of the sign. The advance sign placement distances in table 78 are only suggested values and are not absolute requirements.

Table 78. MUTCD criteria for advance warning sign placement distance.[2]
(Based on MUTCD table II-1)

Posted or 85th percentile speed (mi/h)	Distance from warning sign to potential hazard (ft)[a]						
	Condition A[b] (high judgment needed	Condition B[c] (stop required)	Condition C (deceleration to stated advisory speed [mi/h])				
			10	20	30	40	50
20	175	d	d	-	-	-	-
25	250	d	100	-	-	-	-
30	325	100	150	100	-	-	-
35	400	150	200	175	-	-	-
40	475	225	275	250	175	-	-
45	550	300	350	300	250	-	-
50	625	375	425	400	325	225	-
55	700	450	500	475	400	300	-
60	775	550	575	550	500	400	300
65	850	650	650	625	575	500	375

[a] All distances are based on the assumption that the warning sign is legible to drivers for 125 ft (38 m) in advance of the sign. For large [48-in by 48-in (122-cm by 122-cm)] signs, the legibility distance can be increased to 200 ft (61 m) and each of the entries in this table can therefore be reduced by 75 ft (23 m).
[b] Includes 10.0-s PIEV time but no maneuver time.
[c] Includes 3.0-s PIEV time and allowance for comfortable deceleration rate.
[d] No suggested minimum distance provided; at these speeds, sign location depends on physical conditions at site.
Note: 1 mi = 1.61 km; 1 ft = 0.305 m; 1 in = 2.54 cm



Note: 1 mi = 1.61 km
1 ft = 0.305 m

Figure 47. AASHTO criteria for passenger car deceleration rates at intersections.[1]

2. Critique of Current Design and Operational Criteria

The current criteria for horizontal and vertical placement of signs are not based on any explicit vehicle characteristic. Therefore, the appropriate horizontal and vertical placement of signs is not sensitive to the differences between passenger cars and trucks. However, the potential for blockage of signs by trucks is of concern and this issue is addressed below.

The criteria for longitudinal placement of warning signs depend on the deceleration capabilities of vehicles and the preferences of drivers. The effect of the reduced deceleration and braking capabilities of trucks on longitudinal placement of warning signs is also addressed in the following discussion.

a. Sign Blockage

The potential for blockage of signs by trucks does need to be considered in establishing sign placement criteria. This issue was examined in a 1985 paper which examined both the blockage of roadside signs when a passenger car is passing a truck and the blockage of overhead signs when a passenger car is following a truck.[99]

When a passenger car is passing a truck on the left, the passenger car driver's view of signs on the right side of the roadway is blocked for some distance. The most critical position for the passenger car driver is when the front of his car is even with the rear of the truck. In this position, the passenger car driver's view of roadside signs is blocked for only 150 ft (46 m). Since roadside signs may be legible for more than 150 ft (46 m) and since the passing driver may have had an opportunity to see the same sign while following the truck before he began the passing maneuver, this situation is not critical.

Sign blockage for passenger car drivers does become critical, however, when two or more trucks are traveling together in the right lane. For example, if a second truck is traveling within 63 ft (19 m) in front of the first truck, the passing driver's view is blocked for 455 ft (139 m) from the rear of the first truck. If three trucks are traveling together in the right lane, roadside signs may be blocked for as much as 1,050 ft (320 m).

The potential for obstruction of the view of passing drivers to roadside signs cannot be remedied through changes in the criteria for horizontal and vertical placement of signs, but may require that critical signs be supplemented with overhead signs or with signs placed on the left side of the roadway.

The passenger car driver's view of overhead signs may also be blocked when closely following a truck. When following a truck by five car lengths (95 ft or 29 m), a passenger car driver does not have a full view of an overhead sign mounted with 16 ft (4.9 m) of vertical clearance until the car is within 140 ft (43 m) of the sign. At a speed of 50 mi/h (80 km/h), an overhead sign would be visible to the passenger car driver for only 1.9 s. This situation can be remedied by mounting overhead signs higher or by providing supplementary roadside signs.

b. <u>Longitudinal Placement</u>

The longitudinal placement of advanced warning signs under condition A is based on 10 s of PIEV time. There is no allowance for time to complete a maneuver, such as a lane change, in response to the sign. However, very few decisions on the highway are so complex that they require 10 s of PIEV time for detection and recognition. Therefore, the criteria for condition A certainly include an allowance for decision and response initiation, as in the AASHTO criteria for decision sight distance, and may also implicitly include an allowance for maneuver time. The concepts involved in condition A are very similar to the concepts used in the AASHTO criteria for decision sight distance. The AASHTO decision sight distance criteria in table 37 and the revised criteria for trucks in tables 39 and 40 could be used to derive sign placement criteria for condition A, except that the decision sight distance criteria really only apply to one maneuver type--a lane change approaching a major fork on a freeway

The longitudinal placement of warning signs under condition B is based on the minimum braking distances for dry pavement from the AASHTO Green Book shown in figure 47. The sign placement criteria for condition C are based on the comfortable deceleration rates for passenger car drivers which are also shown in figure 47. These comfortable deceleration rates range from 6.6 to 10.5 ft/s^2 (2.0 to 3.2 m/s^2 or 0.21 to 0.33 g), as shown in table 79. The deceleration rates for condition C are about two-thirds of the rates used for condition B, based on the assumption that drivers will use a lower deceleration rate in slowing down than they would in stopping. While the rates for condition C may be comfortable rates that passenger car drivers would choose on dry pavements, at higher speeds they exceed the deceleration rates that passenger cars can attain in a locked-wheel stop on wet pavement (see deceleration rates used in AASHTO stopping sight distance criteria in table 6). A sensitivity analysis indicating how table 78 should be revised for trucks is presented in the next section.

Table 79. AASHTO criteria for comfortable passenger car deceleration rates.[1]
(Derived from AASHTO Green Book figure II-13)

Speed reached through deceleration (mi/h)	Deceleration rate (ft/s^2)	Deceleration rate (g)
50	10.2	0.32
40	10.3	0.32
30	10.5	0.33
20	7.5	0.23
0	6.6	0.21

Note: 1 mi = 1.61 km
1 ft = 0.305 m

3. Sensitivity Analysis

No sensitivity analyses are needed for the issues of horizontal placement of signs, vertical placement of signs, and sign blockage by trucks. Horizontal and vertical placement criteria for signs are not dependent on the differences between passenger cars and trucks. Sign blockage by trucks must be addressed through sign relocation or placement of supplementary signs.

No formal sensitivity analysis of the criteria for advance placement of warning signs was performed. Instead, the sensitivity analyses performed earlier in this report for stopping sight distance has been adapted to address condition B, since the design condition for condition B is functionally equivalent to stopping sight distance. It would be desirable for the AASHTO and MUTCD criteria for these situations to be consistent. The sign placement criteria for conditions A and C have been revised for trucks in manner consistent with the criteria for condition B, as discussed below. Table 80 presents a revised version of table 78 that is applicable to trucks.

Table 80. Revised criteria for advance warning sign placement distances adequate for trucks.

Posted or 85th percentile speed (mi/h)	Distance from warning sign to potential hazard (ft)[a] Condition A[b] (high judgement needed	Condition B[c] (stop required)	Condition C[d] (deceleration to stated advisory speed [mi/h]) 10	20	30	40	50
20	250	e	e	-	-	-	-
25	325	e	125	e	-	-	-
30	425	175	225	150	-	-	-
35	500	250	325	250	100	-	-
40	600	325	450	375	225	-	-
45	675	425	600	500	325	175	-
50	775	525	750	650	525	325	-
55	850	650	900	825	675	500	225
60	950	775	1,075	1,000	875	675	425
65	1,025	900	1,225	1,200	1,050	850	600

[a] All distances are based on the assumption that the warning sign is legible to drivers for 125 ft (38 m) in advance of the sign. For large [48-in by 48-in (122-cm by 122-cm)] signs, the legibility distance can be increased to 200 ft (61 m) and each of the entries in this table can therefore be reduced by 75 ft (23 m).

[b] Includes 12.0-s PIEV time.

[c] Includes 2.5-s PIEV time and deceleration rates for driver with 70% braking control efficiency driver for consistency with revised stopping sight distance criteria for trucks in table 24.

[d] Based on comfortable deceleration rate equal to two-thirds of the deceleration rate used for condition B.

e No suggested minimum distance provided; at these speeds, sign location depends on physical conditions at site.

Note: 1 mi = 1.61 km
1 ft = 0.305 m
1 in = 2.54 cm

The revised criteria for condition B in table 80 have been based on the deceleration rates used for stopping by a truck with the driver with 70 percent braking control efficiency, as used in the revised sight distance criteria for trucks in table 24. These deceleration rates, which represent a critical condition on wet pavement, would represent very comfortable deceleration rates on dry pavement. The PIEV time has been decreased from 3.0 to 2.5 s for consistency with the AASHTO stopping sight distance criteria. Thus, the sign placement criteria for condition B in table 80 are equal to the candidate stopping sight distance criteria for trucks in table 24 minus the 125-ft (38-m) allowance for the sign legibility distance.

The sign placement criteria for condition C are determined in the same manner as the criteria for condition B except that, for consistency with the passenger car criteria, they are based on comfortable deceleration rates for trucks equal to two-thirds of the deceleration rates used for condition B. This approach to condition C results in relatively long advance warning sign placement criteria for trucks, because of their relatively low deceleration capabilities, but maintains the concept that truck drivers would use lower deceleration rates in slowing than in braking to a stop.

The sign placement criteria for conditions B and C in table 80 are based on deceleration rates for trucks with conventional brake systems and a relatively poor-performance driver (i.e., 70 percent driver control efficiency). If trucks with antilock brake systems come into nearly universal use, the current MUTCD sign placement criteria in table 78 will accommodate trucks.

For condition A in table 80, the PIEV time has been increased from 10 to 12 s. This was done primarily so that all of the criteria for condition A would be greater than the criteria for condition B, which would not be the case if the 10-s PIEV time had been maintained. However, the additional time increment for trucks could be considered to represent all or part of their higher maneuver times.

The recommended advance warning sign placement criteria for trucks are 21 to 43 percent longer than the current MUTCD criteria for situations where high judgment is required (condition A), 38 to 75 percent longer where a complete stop is required (condition B), and 25 to 92 percent longer where deceleration to a stated advisory speed is required (condition C).

4. Recommended Revisions to Highway Design and Operational Criteria

Revised advance warning sign distances have been developed for trucks, as shown in table 80. The recommended advance warning sign distances for trucks can be reduced if trucks with antilock brakes come into widespread use.

Adoption of the recommended advance warning sign placement criteria for trucks would probably be very cost effective. Table 81 shows the percentage reduction in truck accidents required for cost effectiveness of sign relocation to implement revised advance warning sign placement criteria. The table is based on a replacement cost of $60/sign and assumes that relocating an advance warning sign has the potential to reduce truck accidents over 1,000 ft (305 m) of a rural two-lane highway. The table shows that the percentage accident reduction in truck accidents required for cost effectiveness is always less than 5 percent and, in most cases, is less than 1 percent. Therefore, adoption of the revised advance warning sign criteria in table 80 is recommended both because they are potentially cost effective for trucks and because they are based on models consistent with established AASHTO stopping sight distance and decision sight distance criteria.

Table 81. Minimum percent reduction in truck accidents required for cost effectiveness of sign relocation to implement revised advance warning sign distances.

	Minimum percent reduction in truck accidents				
	Percent trucks				
(veh/day)	1%	5%	10%	20%	30%
1,000	4.9	1.0	0.5	0.2	0.2
2,000	2.8	0.6	0.3	0.1	0.1
3,000	1.9	0.4	0.2	0.1	0.1
4,000	1.5	0.3	0.1	0.1	0.05
5,000	1.2	0.2	0.1	0.1	0.04
6,000	1.0	0.2	0.1	0.1	0.03
7,000	0.9	0.2	0.1	0.04	0.03
8,000	0.8	0.2	0.1	0.04	0.03
9,000	0.7	0.1	0.1	0.03	0.02
10,000	0.6	0.1	0.1	0.03	0.02
11,000	0.6	0.1	0.1	0.03	0.02
12,000	0.5	0.1	0.1	0.03	0.02
13,000	0.5	0.1	0.05	0.02	0.02
14,000	0.5	0.1	0.05	0.02	0.02
15,000	0.4	0.1	0.04	0.02	0.01

IV. CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations developed in the study are summarized below.

A. Design Vehicles

1. The WB-50 design vehicle given in the 1984 AASHTO Green Book, which has a 37-ft (11.3-m) semitrailer, should be replaced with a larger vehicle with a 45-ft (13.7-m) semitrailer for use off of the designated highway system.

2. Two STAA design vehicles should be added to the Green Book:

   - STAA single with 48-ft (14.6-m) semitrailer.

   - STAA double with two 28-ft (8.5-m) trailers.

3. A design vehicle with a 53-ft (16.1-m) semitrailer should be added to the Green Book for use where permitted by State law or STAA "grandfather" provisions. This design vehicle may become appropriate for more general use in the 1990's.

4. Longer combinations vehicles (LCVs) are not appropriate for general use as design vehicles at this time, but could be added to the Green Book for use in those States where they are permitted.

B. Stopping Sight Distance

5. Current AASHTO stopping sight distance criteria are adequate for trucks with antilock brake systems.

6. Current AASHTO criteria are adequate at vertical sight restrictions for trucks with the conventional brake systems and the best performance driver. At horizontal sight restrictions, a truck with the best performance driver needs approximately 50 ft (15 m) of additional stopping sight distance.

7. Current AASHTO criteria are not adequate to accommodate trucks with conventional braking systems and poor performance drivers. Many drivers have little experience with the proper procedures for controlled braking in emergency situations because emergency situations on the road are rare events and very few drivers have had the opportunity to practice emergency stops on a test track. A driver with 70 percent control efficiency (a poor, but not extreme value) requires 25 to 425 ft (8 to 130 m) of additional stopping sight distance, depending on speed. The higher driver eye height for trucks offsets some, but not all, of this difference at vertical sight restrictions.

8. Candidate stopping sight distance criteria to accommodate trucks with conventional brake systems are given in table 24. These criteria are cost effective only for new construction or major reconstruction projects on rural two-lane highways that carry more than 800 trucks/day and rural freeways that carry more than 4,000 trucks/day. These criteria are not cost effective for rehabilitation projects and will not be needed if antilock brake systems for trucks are required by Government regulations or come into widespread use.

C. Passing and No-Passing Zones on Two-Lane Highways

9. The passing sight distance requirements for a passenger car passing another passenger car based on a model recently developed by Glennon are in good agreement with the current MUTCD criteria. The AASHTO criteria for passing sight distance have an entirely different basis and are much more conservative.

10. Passing scenarios involving a passenger car passing a truck, a truck passing a passenger car, and a truck passing a truck require progressively more passing sight distance than a passenger car passing a passenger car. Candidate passing sight distance criteria for each of these situations are given in tables 33 and 34.

11. Since passing maneuvers involving trucks require longer sight distance than passing maneuvers involving just passenger cars, they also require longer vertical curves if a passing zone is to be maintained over a crest. For example, at 70 mi/h (113 km/h) a passing maneuver involving a truck may require a vertical curve 300 to 500 ft (90 to 150 m) longer than a passenger car passing a passenger car. However, a truck can safely pass a passenger car on any vertical curve where a passenger car can safely pass a truck.

12. There are no current criteria for passing zone lengths, except for the default 400-ft (122-m) guideline set by the MUTCD. For all design speeds above 30 mi/h (48 km/h), the distance required for one vehicle to pass another is substantially longer than 400 ft (122 m), indicating the need for longer passing zones. The required passing distances are increased substantially when the passing vehicle, the passed vehicle, or both, are trucks, as shown in table 36.

13. Changes in passing sight distance criteria to accommodate a truck as the passing vehicle may not be needed because most passing zones on two-lane highways are not long enough to accommodate delayed passes by trucks. However, trucks may be able to complete flying passes (i.e., without slowing down) in relatively short zones. Changes in passing sight distance criteria to accommodate trucks should not be considered without an operational analysis of the reduction in the level of service on two-lane highways that would result from eliminating or shortening passing zones where it is safe for a passenger car to pass another passenger car.

D. Decision Sight Distance

14. Trucks may require 100 to 400 ft (30 to 122 m) more decision sight distance than passenger cars at a design speed of 70 mi/h (113 km/h), and lesser amounts of additional decision sight distance at lower design speeds.

15. The higher driver eye height for trucks offsets the increased decision sight distance requirement in most cases at vertical sight restrictions, but not at horizontal sight restrictions.

16. A change in decision sight distance criteria to accommodate trucks by using longer vertical curves on the approach to major decision points would be cost effective only in unusual situations with extremely high accident rates. Such improvements are cost effective only if they provide accident reduction benefits 4 times those found for stopping sight distance improvements for trucks. It is unlikely that accident reduction benefits that large could be expected. Therefore, no change in existing decision sight distance criteria is recommended.

E. Intersection Sight Distance

17. For intersections with no control (case I), trucks may require up to 69 percent more sight distance than passenger cars, but the amount of increased sight distance needed by trucks is highly related to the relative speeds of the approaching vehicles.

18. For intersections with YIELD control (case II), the intersection sight distance requirements for trucks are the same as the stopping sight distance requirements (see table 24). Where a clear sight triangle with adequate stopping sight distance is not provided, the sight distance requirements can be lowered by placing advisory speed limit signs on the approaches. A cost-effectiveness analysis showed that providing additional sight distance by clearing the sight triangle in each quadrant of an intersection can be very cost effective when the clearing cost is relatively small (e.g., $1,000 per intersection). On the other hand, very expensive clearing operations (e.g., removing structures or embankments) are almost never cost effective.

19. Based on the Gillespie model for intersection clearance times, the larger trucks currently on the road require up to 17.5 percent more sight distance for an intersection crossing maneuver (case III-A) than the current AASHTO criteria based on a WB-50 truck.

20. For left- and right-turn maneuvers at intersections (cases III-B and III-C), use of truck characteristics in the current AASHTO models for curves B-2a & Ca and B-2b & Cb can require sight distances up to 139 percent greater than a passenger car. The AASHTO models can require over 3,000 ft (900 m) of sight distance at design speeds above 50 mi/h (80 km/h). Very few intersections have such long sight distances available, and it is unlikely that drivers could judge the location and speed of an oncoming vehicle even if they were available. Rather, this result indicates that the current AASHTO model is unrealistic and needs to be revised.

21. Several alternative models for intersection sight distance (see figures 33 and 34) were developed based on data from the literature and pilot field studies. These field studies have demonstrated a methodology to collect data concerning the intersection sight distance requirements of passenger cars and trucks. In particular, an intersection sight distance model based on gap acceptance should be considered. A full-scale study of this issue is recommended.

F. Intersection and Channelization Geometrics

22. Intersection and channelization geometrics should be based on the low-speed offtracking characteristics of the larger design vehicles identified above. The offtracking characteristics of these vehicles are documented in section III E and in appendix C of volume II.

G. Railroad-Highway Grade Crossing Sight Distance

23. The current FHWA and AASHTO criteria for the sight distance along the highway ahead to a railroad crossing should be increased by up to 54 percent, depending on the design speed, for trucks with conventional brake systems. No changes in the criteria are needed to accommodate a truck with an antilock brake system.

24. The current FHWA and AASHTO criteria for sight distance along the railroad tracks for a moving vehicle should be increased by up to 49 percent, depending upon design speed for trucks with conventional brake systems. Current criteria are adequate for trucks with antilock brake systems.

25. The current FHWA and AASHTO criteria for sight distance along the railroad tracks for a stopped vehicle are adequate to accommodate trucks.

H. Crest Vertical Curve Length

26. See conclusions for stopping sight distance, passing sight distance, and decision sight distance.

I. Sag Vertical Curve Length

27. Trucks with antilock brake systems require shorter sag vertical curve lengths than current AASHTO criteria.

28. Trucks with conventional brake systems may require sag vertical curves up to 670 ft (200 m) longer than current AASHTO criteria.

29. The current AASHTO criteria may be meaningless at higher speeds because vehicle headlights do not illuminate the roadway for the full stopping sight distance. The AASHTO model for sag vertical curve length needs to be fully reexamined because the rationale for the connection between headlight beam distance and sag vertical curve length appears to be outdated.

J. Critical Length of Grade

30. Based on recent field data, the AASHTO criterion for truck weight-to-power ratio used to define the critical length of grade should be reduced from 300 lb/hp (0.18 kg/W) to 250 lb/hp (0.15 kg/W). The current 10 mi/h (16 km/h) speed reduction criterion should be retained.

31. Implementation of recommendation 30 should be deferred pending action on the "Turner truck" proposal, which could substantially increase the weights of trucks on the highways. Some of the effect of gross vehicle weight increases up to 150,000 lb (68,000 kg) will undoubtedly be offset by use of more powerful tractors, and technological advances in engine size could eventually compensate for all of the increase.

K. Lane Width

32. The current AASHTO lane width criteria are adequate to accommodate trucks.

L. Horizontal Curve Radius and Superelevation

33. Current AASHTO criteria for horizontal curve radius and superelevation at particular design speeds are adequate to accommodate trucks. The existing criteria provide margins of safety against

skidding off the road and against rollover that are substantially lower for trucks than for passenger cars. However, the existing AASHTO criteria provide an adequate margin of safety for a truck if the truck is traveling at the design speed.

34. Current superelevation transition methods appear adequate to accommodate trucks. Use of spiral transitions is preferable to the traditional 2/3-1/3 rule, but the resulting reduction in maximum lateral acceleration is typically only about 0.01 g.

35. Increased emphasis is needed on the realistic selection of design speeds for horizontal curves, particularly on freeway ramps. It is critical that the design speeds selected for off-ramps are consistent with the design speed of the mainline roadway. It is recommended that the lower range values of ramp design speed in table X-1 of the Green Book not be used for roadways that carry substantial volumes of truck traffic.

M. Pavement Widening on Horizontal Curves

36. Revised criteria for pavement widening on horizontal curves to accommodate an STAA single 48-ft (14.6-m) semitrailer truck are given in table 75. The revised criteria are expressed in terms of minimum lane widths on horizontal curves rather than specified amounts of pavement widening. These criteria are adequate for vehicles traveling up to the design speed and incorporate consideration of both high-speed and low-speed offtracking based on a new offtracking model developed in the present study.

N. Cross-Slope Breaks

37. No data are available to determine the adequacy for trucks of the current AASHTO criteria for pavement/shoulder cross-slope breaks.

38. Cross-slope breaks at the centerline crown of a highway should be kept to a minimum to maintain safe truck operations in passing maneuvers.

O. Roadside Slopes

39. No data are available concerning the adequacy for trucks of current roadside slope design criteria.

P. Vehicle Change Interval

40. Trucks require vehicle change intervals between 40 and 110 percent longer than passenger cars, depending on approach speed, approach grade, and intersection width. However, the existing guidelines for vehicle change interval in the FHWA Traffic Control Devices Handbook should not be revised without an analysis to assess the extent of operational and safety problems that would be created by reduced levels of service at intersections.

Q. Sign Placement

41. The MUTCD advance warning sign placement criteria for condition B (stop required) should be made consistent with similar concepts used in AASHTO sight distance design criteria. The criteria for conditions A and C should be adjusted for consistency with the revised criteria for condition B.

42. Advance warning sign placement criteria for trucks with conventional brake systems should be longer than the current criteria which are based on consideration of passenger cars. The advance warning sign placement distances for condition A (high judgment required) should be increased by 75 to 175 ft (23 to 53 m), depending on prevailing speed, to accommodate trucks. The advance warning sign placement distance for condition B (stop required) should be increased by 75 to 250 ft (23 to 76 m), depending on prevailing speed, to accommodate trucks. The criteria for condition C (deceleration to a stated advisory speed) should be increased by 50 to 575 ft (23 to 175 m), depending upon prevailing speed and posted advisory speed, to accommodate trucks. These recommended changes in advance warning sign placement criteria would not be necessary if trucks with antilock brake systems come into nearly universal use.

43. Implementation of revised warning sign placement criteria would be cost effective under nearly all conditions even if this required moving or replacing signs.

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