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16. Abstract

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GEOMETRIC DESIGN CONSIDERATIONS FOR SEPARATE TRUCK LANES

by

John M. Mason

Robert C. Briggs

and

Kay L. Schwartz

Research Report 331-1
Research Study Number 2-8-84-331
Geometric Design Considerations for Separate Truck Lanes

Sponsored by

Texas State Department of Highways and Public Transportation
in cooperation with the
U. S. Department of Transportation, Federal Highway Administration

April 1986

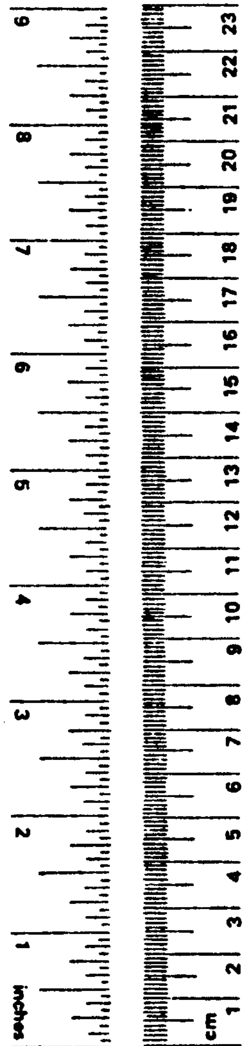
TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

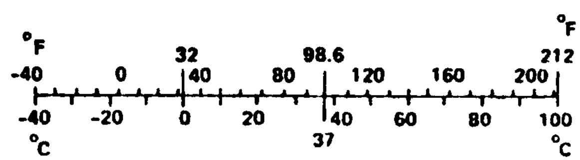
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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This phase of the project was sponsored by the Texas State Department of Highways and Public Transportation and Federal Highway Administration. Dock Burke served as the Principal Investigator. John M. Mason was the Principal Staff Engineer. Robert Briggs and Kay Schwartz served as Graduate Research Assistants. Bob Guinn was the Contact Representative for the State Department of Highways and Public Transportation. The cooperation of these individuals and the assistance of the Department made the collection of this information possible.

ABSTRACT

This report examines past truck related research to determine the applicability of current geometric design policies to special truck lane facilities. Recommendations are made to help fill the voids in existing design policy. The policies addressed include vehicle characteristics, sight distance, horizontal and vertical alignment, and cross section elements. The report describes specific design elements, discusses their appropriateness to special truck lane facilities, and recommends alternative design criteria where past research warrants possible changes.

EXECUTIVE SUMMARY

One alternative for improving safety and capacity along heavily traveled truck corridors is to provide an exclusive truck facility. These facilities provide a means of isolating and separating automobiles from large trucks. The segregation may be in the form of separate truck lanes or exclusive truck travelway sections in the median or along independent rights-of-way. The purpose of this study was to investigate the feasibility of providing such facilities in the median area. The four main elements examined in this report were: vehicle characteristics, sight distance, horizontal alignment, and cross section elements.

When vehicle characteristics were reviewed, special attention was given to the vehicle's height, width, length, driver eye height, vehicle headlight height, weight-to-horsepower ratio, and truck braking distances. The current maximum height for large trucks is 13.5 ft. After reviewing the design criteria necessary for exclusive truck facilities, there is no change necessary. The width of design vehicles (102 in.) is set by the 1982 Surface Transportation Assistance Act. A maximum length of vehicle was assumed to be 65 ft.

The current AASHTO value for driver eye height of 3.5 ft is not representative of trucks. The current weight-to-horsepower ratio of 300 pounds is a reasonable estimate of horsepower characteristics of heavy vehicles. Since research has shown that cars are able to stop in two-thirds the distance required by heavy trucks, the braking distance equation has been modified to represent heavy trucks.

When reviewing the sight distance elements necessary for exclusive truck facilities, the perception-reaction time may need to be increased to 3.2 seconds. With the increased stopping distance requirements and the recommended 3.2 seconds driver perception-reaction time, the current AASHTO stopping sight distance values are low. However, in the design of crest vertical curves, truck driver eye height above (approximately) 5 feet does compensate for increased truck braking distances. If future truck configurations result in lower driver eye height, the combined effect of increases in perception-reaction time and longer stopping distances will require further investigation.

AASHTO design criteria for horizontal curves for high-speed facilities appear to be adequate for exclusive truck facilities design. Vertical curves lengths have been calculated for increased stopping sight distance requirements.

The AASHTO recommendations for high-speed facilities concerning lane widths, shoulder widths, and sideslopes are applicable for design of exclusive truck facilities. Guardrails, however, need additional study to ensure sufficient strength in redirecting errant heavy vehicles.

IMPLEMENTATION STATEMENT

The information assembled in this report is the result of a synthesis of truck related research and current AASHTO guidelines. Since no field operation data were collected, the conclusions are the result of a critical review of existing literature. Interpretation of the findings are based on information available during the conduct of study. The goal of these efforts was to establish design guidelines for truck facilities.

This report has been organized for ready reference to traditional geometric design elements. Vehicle characteristics, sight distances, horizontal alignment, vertical alignment, and cross section elements comprise the major sections. Each basic design element was then classified into its components for additional detailed discussion.

The findings of this investigation provide an initial basis for examining the feasibility of constructing truck facilities within existing highway corridors. Further, the report represents current knowledge relative to truck lane and/or separate facilities.

DISCLAIMER

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INTRODUCTION

The rapid growth in traffic on the Texas highway system has prompted the State Department of Highways and Public Transportation (SDHPT) to examine various techniques of handling the simultaneous increase in truck traffic demands. The Texas SDHPT has decided to evaluate special truck lane needs along the I-35 corridor between Dallas-Ft. Worth and San Antonio. The overall objectives of this study were to identify areas of high truck volumes, to establish operational and design procedures to deal with truck traffic, and to evaluate the corridor and systemwide effects of the proposed recommendations.

One alternative of particular interest is the feasibility of using existing median rights-of-way for an exclusive truck lane facility. The I-35 corridor was selected as the first segment for evaluation. Findings of this initial study can be used to establish basic design elements for evaluating other candidate corridors in the State.

The analysis procedure involved two distinct phases. The first phase documented the physical problems associated with placing exclusive truck facilities (ETFs) in the existing rights-of-way. The second phase consists of the review of current geometric design policy to determine applicability to ETFs. Major elements of the study included geometrics, right-of-way availability, operations, safety, pavement requirements, and costs of the potential improvements.

Roadway geometry was the critical element in the analysis. Since geometric design effects right-of-way limits, operational efficiency, relative safety, and construction costs, the establishment of geometric requirements of the system were necessary before other elements could be properly addressed.

Current geometric design policies of the SDHPT reflect the policies outlined in AASHTO's Redbook and Bluebook. However, these policies assume that the majority of the design traffic will be automobiles with a small percentage of large trucks.

No publication exists that provides specific guidelines for the geometric design of exclusive truck facilities. A detailed literature review was conducted to determine the feasibility of applying the findings to the design of ETFs. This report contains the review of the pertinent design elements and recommends additions to fill the voids in existing design policies. The following elements were examined: vehicle characteristics, sight distance, horizontal alignment, vertical alignment, and cross section elements.

DESIGN VEHICLES

The geometric design of the roadway is influenced both by the physical and operational characteristics of the intended vehicles. AASHTO (1) uses the design vehicle approach, where all vehicles utilizing the facilities are examined and grouped into classes of similar operational and physical characteristics. Then, a "critical" design vehicle selected. This design vehicle is typically one which has the largest overall dimensions, weight, or turning radius. By identifying these specific characteristics and selecting the vehicle type with the most severe attributes, it is assumed that any other smaller vehicles will be accommodated.

Leisch and Associates (2) have tabulated the relationship of geometric design elements to vehicle characteristics. Table 1 summarizes vehicle characteristics and their related geometric design elements. This serves as a suitable guide to geometric design of the roadway as a function of vehicle characteristics.

VEHICLE CHARACTERISTICS

The vehicle characteristics of interest in truck facilities design are vehicle length, width, height (including center of gravity), height of eye and headlights, weight-to-horsepower ratios, and braking characteristics. AASHTO provides for two general classes of vehicles, cars and trucks. The truck class consists of four design vehicle types: the single-unit truck (SU), the intermediate semi-trailer (WB-40), the large semi-trailer (WB-50), and the "Double Bottom" semitrailer-full trailer combination (WB-60). AASHTO design vehicle dimensions and a summary table of design vehicle turning radii are shown in Table 2.

AASHTO also provides for minimum turning radii for each of its design vehicles. Diagrams of the "swept path" of each vehicle are shown in Figures 1 through 3. Note that AASHTO does not provide information on center of gravities, vehicle headlight heights and driver eye heights, weight/hp ratios, or braking characteristics for each of its design vehicles. Acceleration characteristics are given for passenger cars with minor attention given to trucks. The eye height criteria used by AASHTO is a 3.5 ft driver eye height which is representative of the passenger car class. This could be used as a conservative estimate for trucks; however, AASHTO assumes that in most cases this conservative estimate compensates for increased braking distances required by trucks. This assumption is not always true and is discussed later under the formulation of stopping sight distance.

AASHTO does not include a triple-trailer combination in its array of design vehicles. Yu and Walton (3), in a study of the characteristics of double and triple combinations in the U.S., identified six major types of combinations in operation from 1966 to 1980. These combinations are shown in Figure 4. Of these six types, only two are represented by AASHTO design vehicles. The two-axle tractor and tandem axle semitrailer, both are referred to as the "Western Double" configuration. If the enactment of the 1982 Surface Transportation Assistance Act represents a trend toward longer, heavier trucks, the use of these types of doubles and triples may become more widespread. Therefore representative design vehicles for different types of combinations will be needed in order to include these trucks in the design of exclusive truck facilities.

Vehicle Heights

The design vehicle heights are consistent at 13.5 ft. Most states restrict vehicle heights to this value; there is no indication that it will change.

GEOMETRIC FEATURE	RELATED VEHICLE CHARACTERISTIC
SIGHT DISTANCE	
Stopping Sight Distance	Braking Distance Eye Height
Passing Sight Distance	Vehicle Length Acceleration
HORIZONTAL ALIGNMENT	
Superelevation	Vehicle Height (C.G.)
Degree of Curve	Vehicle Height (C.G.)
Widths of Turning Roadways	Vehicle Length Vehicle Width
Pavement Widening on Curves	Vehicle Length Vehicle Width
VERTICAL ALIGNMENT	
Maximum Grade	Weight to Horsepower ratio
Critical Length of Grade	Weight to Horsepower ratio
Climbing Lanes	Weight to Horsepower ratio
Vertical Curves	Eye and Headlight Heights
Vertical Clearance	Vehicle Height
CROSS SECTION ELEMENTS	
Lane Widths	Vehicle Width
Shoulder Widths	Vehicle Width
Traffic Barriers	Vehicle Mass and C.G.
Side Slopes	Vehicle Height (C.G.)

Table 1. Geometric Features and Related Vehicle Characteristics (2).

Design Vehicle Type	Symbol	Dimension (ft)									
		Overall			Overhang						
		Height	Width	Length	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	18		
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 axel to the hitch point.
T is the distance from the hitch point to the lead effective axel of the following unit.

Design Vehicle Type	Passenger Car	Single Unit Truck	Single Unit Bus	Articulated Bus	Semitrailer Intermediate	Semitrailer Combination Large	Semitrailer Full-Trailer Combination	Motor Home	Passenger Car with Travel Trailer	Passenger Car with Boat and Trailer
Symbol	P	SU	BUS	A-BUS	WB-40	WB-50	WB-60	MH	P/T	P/B
Minimum turning radius (ft)	24	42	42	38	40	45	45	42	24	24
Minimum inside radius (ft)	15.3	28.4	23.2	21.0	19.9	19.8	22.5	28.4	5.5	10

Minimum turning radii of design vehicles.

Table 2. AASHTO Design Vehicle Dimensions and Minimum Turning Radii (1).

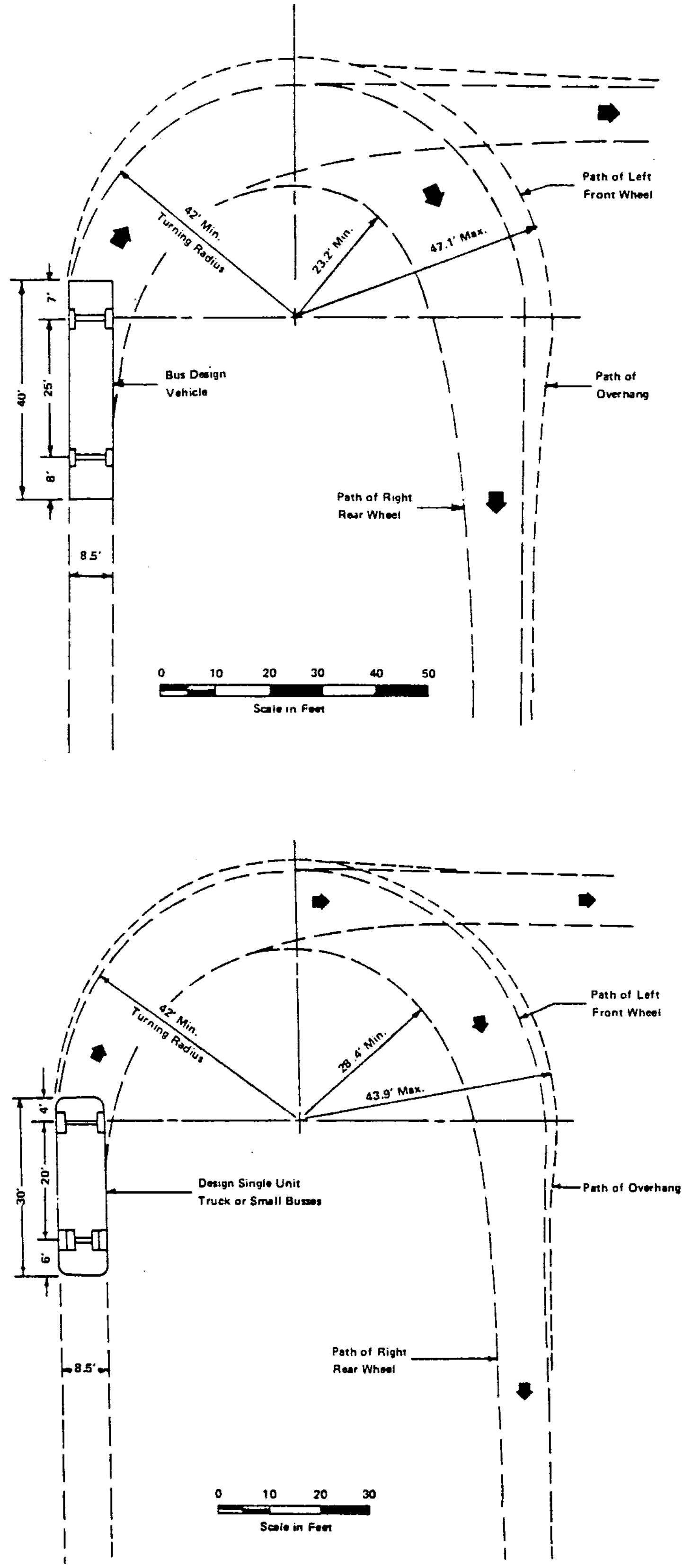
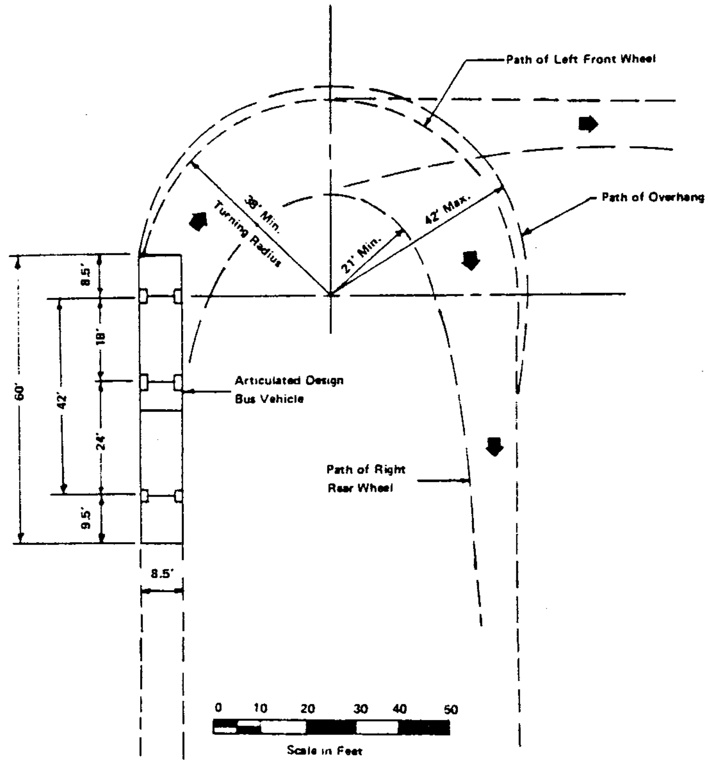


Figure 1. The "Swept Path" of the Design Vehicles (1).



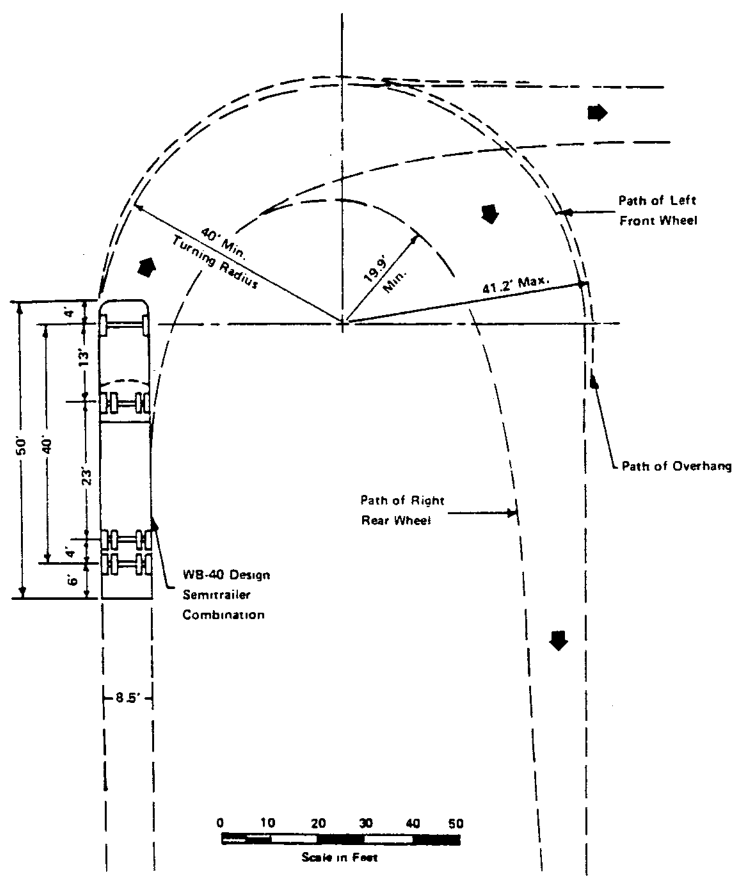


Figure 2. The "Swept Path" of the Design Vehicles (1).

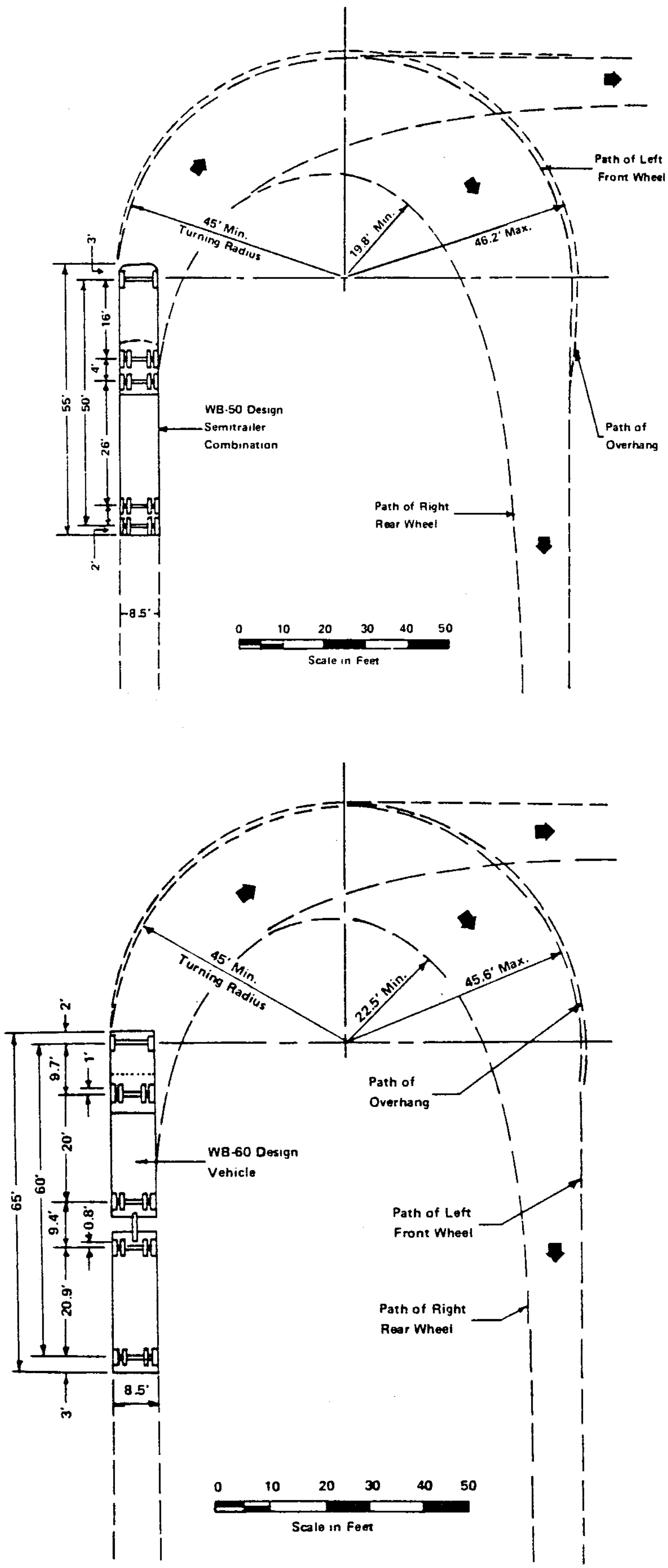
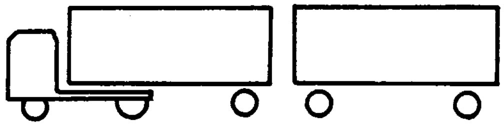
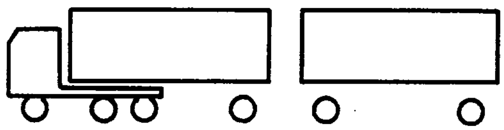


Figure 3. The "Swept Path" of the Design Vehicles (1).



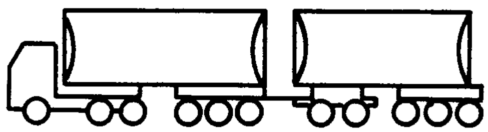
Sample Size: 48,482 (84%)
Avg. Wheelbase: 59 Feet
Avg. Weight: 56,390 Lbs.

Western Double (Two-Axle Tractor)



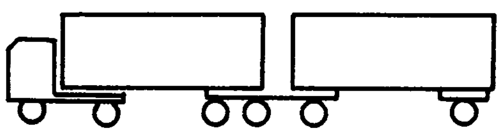
Sample Size: 4,983 (8.6%)
Avg. Wheelbase: 60 Feet
Avg. Weight: 60,850 Lbs.

Western Double (Three-Axle Tractor)



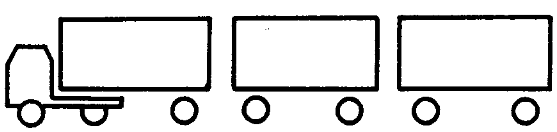
Sample Size: 819 (1.4%)
Avg. Wheelbase: 53 Feet
Avg. Weight: 94,360 Lbs.

Michigan Double Tanker Truck



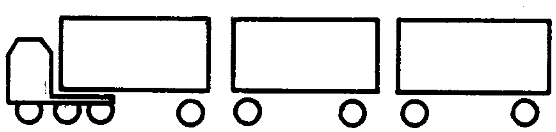
Sample Size: 689 (1.2%)
Avg. Wheelbase: 54 Feet
Avg. Weight: 56,890 Lbs.

Western Double (Tandem Axle Semi-Trailer)



Sample Size: 260
Avg. Wheelbase: 89 Feet
Avg. Weight: 81,870 Lbs.

Triple Trailer Combination (Two-Axle Tractor)



Sample Size: 83
Avg. Wheelbase: 91 Feet
Avg. Weight: 82,670 Lbs.

Triple Trailer Combination (Three-Axle Tractor)

Figure 4. Characteristics of Double and Triple Combinations in the U.S. (3).

Whiteside et al. (4) state that vehicle heights are primarily a function of loading practice, overhead clearances on the highway, and the effect of vehicle height on traffic. Except for certain specialized cargo, the truck transport industry has shown little interest in raising the legal standard for vehicle heights. Changes in vehicle heights are impractical given current cargo stacking limitations, freight depot design, and warehouse loading dock heights. Increase in vehicle heights would also adversely affect vehicle stability on curves and in high crosswinds.

Table 3 (5) lists legal heights of motor vehicles as of 1980. Of the 50 states, 11 had maximum heights above 13.5 ft.

Highway construction costs are directly related to maximum vehicle height regulations. Overhead clearances of bridges, utilities, underpasses, traffic control devices and signs are each controlled by the height of the design vehicle. Superelevation and superelevation transition are influenced by vehicle heights. An increase in vehicle center of gravity introduces a higher probability of overturning. The sensitivity of loaded and unloaded trucks has not been sufficiently investigated. Current practices of private industry, highway departments, and vehicle manufacturers do not suggest any changes in the 13.5 ft limitation.

Vehicle Widths

AASHTO assumes a 102-inch width for all truck type design vehicles. This is in accordance with the 1982 Surface Transportation Assistance Act which permits the operation of 102-inch vehicles on the designated system. Therefore, as a minimum design standard, a 102-inch vehicle width is required for geometric design of ETFs.

Vehicle widths affect lane widths, shoulder widths, widths of turning roadways, pavement widening on curves, and horizontal clearances on bridges and in tunnels. The lateral placement of vehicles in the traveled lane is critical, especially during overtaking and passing maneuvers on two-lane highways. These considerations must be carefully assessed since as the vehicle width increases, so will construction costs.

If a design vehicle width is adopted which is not representative of the actual vehicle widths on the facility, serious problems will arise. Underestimating actual vehicle widths leads to the design of inadequate lane widths and insufficient allowance for safe lateral clearances to obstructions. Overestimating actual vehicle widths leads to the design of excessively wide traffic lanes and needless construction expenditures.

One solution to the above dilemma would be to review the states' overwide permit issuance along the corridor in question for a number of years prior to the construction of facilities. A representative sample of oversize vehicle permits issued in the past could be used to approximate the number and magnitude of widths of oversize vehicles using the corridor. From this sample, an 85th percentile vehicle width could be approximated and used for design purposes.

This type of review process would not be feasible at this time since records of permit issuance are currently documented on paper. Many permits are issued each year and analysis of several years' permits would be cost

STATE	HEIGHT LIMIT (FT)
ARIZONA	14
CALIFORNIA	14
COLORADO	14
DISTRICT OF COLUMBIA	12.5
IDAHO	14
MAINE	14
MONTANA	14
NEBRASKA	14
NEVADA	14.5
UTAH	14
WASHINGTON	14
WYOMING	14
ALL OTHER STATES	13.5

Table 3. Legal Heights of Motor Vehicles (1980) (5).

prohibitive and labor intensive. However, future permit issuance procedures of the SDHPT will be automated with issued permits stored on tape files. It may then become possible to search several years' permits and produce statistical data regarding oversize vehicle operation in the State.

Vehicle Lengths

AASHTO design vehicle lengths are as follows: single unit truck - 30 ft, single unit bus - 40 ft, articulated bus - 60 ft, intermediate semitrailer - 55 ft, and "Double Bottom" semitrailer-fulltrailer - 65 ft.

With the exception of over-length loads, these design vehicles represent the major portion of truck types being operated in the State of Texas.

AASHTO does not provide a design vehicle for the 105-ft double and triple combinations. These types of trucks have been used for years in the western states, particularly Oregon, Nevada, and Utah. Idaho has also allowed 98 ft lengths; Montana and Wyoming, 85 ft and; South Dakota has allowed lengths of up to 80 ft. All other states have had length limitations of 65 ft or below (6).

The Surface Transportation Assistance Act provides for the following (7):

> On the Interstate System and on primary system highways designated by the Secretary, no state may impose length limitations less than 48 feet on a semitrailer unit operating in a truck-tractor semitrailer combination or less than 28 feet on a semitrailer or trailer operating in a truck tractor - semitrailer - trailer combination. No state may reduce length limitations which were in effect in that state on December 1, 1982.
>
> On the Interstate system and on primary system highways designated by the Secretary, no state may prohibit the use of combinations consisting of a truck tractor and two trailing units.

The governor of Texas signed into law in 1983 House Bill No. 1601 which reads as follows:

> No motor vehicle, other than a truck-tractor, shall exceed a length of forty-five (45) feet. Except as provided in Subsection (C-1) of this section, it shall be lawful for any combination of not more than three vehicles to be coupled together including, but not limited to, a truck and semitrailer, truck and trailer, truck tractor and semitrailer and trailer, or a truck-tractor and two trailers, provided such combination of vehicles, other than a truck-tractor combination, shall not exceed a length of 65 feet unless such vehicle or combination of vehicles is operated exclusively within the limits of an incorporated city or town; and unless, in the case of any combination of such vehicles, same be operated by municipal corporations in adjoining suburbs

wherein said municipal corporation has therefore been using such or like equipment in connection with an established service to such suburbs of the municipality; provided further, that motor buses as defined in Acts of the Forty- First Legislature, Second Called Session, 1929, Chapter 88, as amended, exceeding 35 feet in length, but not exceeding 40 feet in length, may be lawfully operated over the highways of this state if such motor buses are equipped with air brakes and have a minimum of four tires on the rear axle; and provided further, that the above limitations shall not apply to any mobile home or to any combination of mobile home and a motor vehicle, but no mobile home and motor vehicle combination shall exceed a total length of 55 feet.

A semitrailer may not exceed a length of 57 feet when operated in a truck-tractor and semitrailer combination. A semitrailer or trailer may not exceed a length of 28 1/2 feet when operated in a truck tractor, semitrailer, and trailer combination.

The length limitations in this subsection do not include any safety device determined by regulation of the Department of Transportation or by rule of the Department of Public Safety to be necessary for the safe and efficient operation of motor vehicles.

The length limitations in this subsection for semitrailers and trailers do not apply to semitrailers or trailers that were being actually and lawfully operated in this state on December 1, 1982.

Note that the Federal Legislation seeks to provide minimum length standards while the state legislation imposes maximums. Thus no state may impose length standards or specify a maximum number of towed units below that specified by the Federal Government. Therefore the State standards, or those specified by House Bill 1601 should be used to establish design vehicle lengths for use in design. H.B. 1601 allows for a 45-ft length for a single unit truck, while AASHTO's design single unit truck length is 30 ft. The State length limitations for busses coincides with AASHTO criteria. AASHTO provides a WB-50 to represent a truck-semitrailer combination. The overall length of this design vehicle is 55 ft. State limits provide for a 65 ft tractor-semitrailer combination with trailer lengths up to 57 ft. As a result, the AASHTO design vehicles do not adequately represent the single unit truck, or the tractor-semitrailer combination from the length point of view (see Figure 5). This is important since vehicle lengths will determine the offtracking characteristics of the vehicles, which in turn affect pavement widths. If AASHTO design vehicles are used for design, inadequate pavement widths on turns and intersections will result. Other AASHTO design vehicles should adequately represent existing vehicle types.

Consideration may need to be given to designing truck facilities to accommodate the 105-ft double and triple combinations mentioned earlier. Although State law prohibits operations of these truck types at this time, future conditions such as fuel shortages may cause a relaxation of these restrictions.

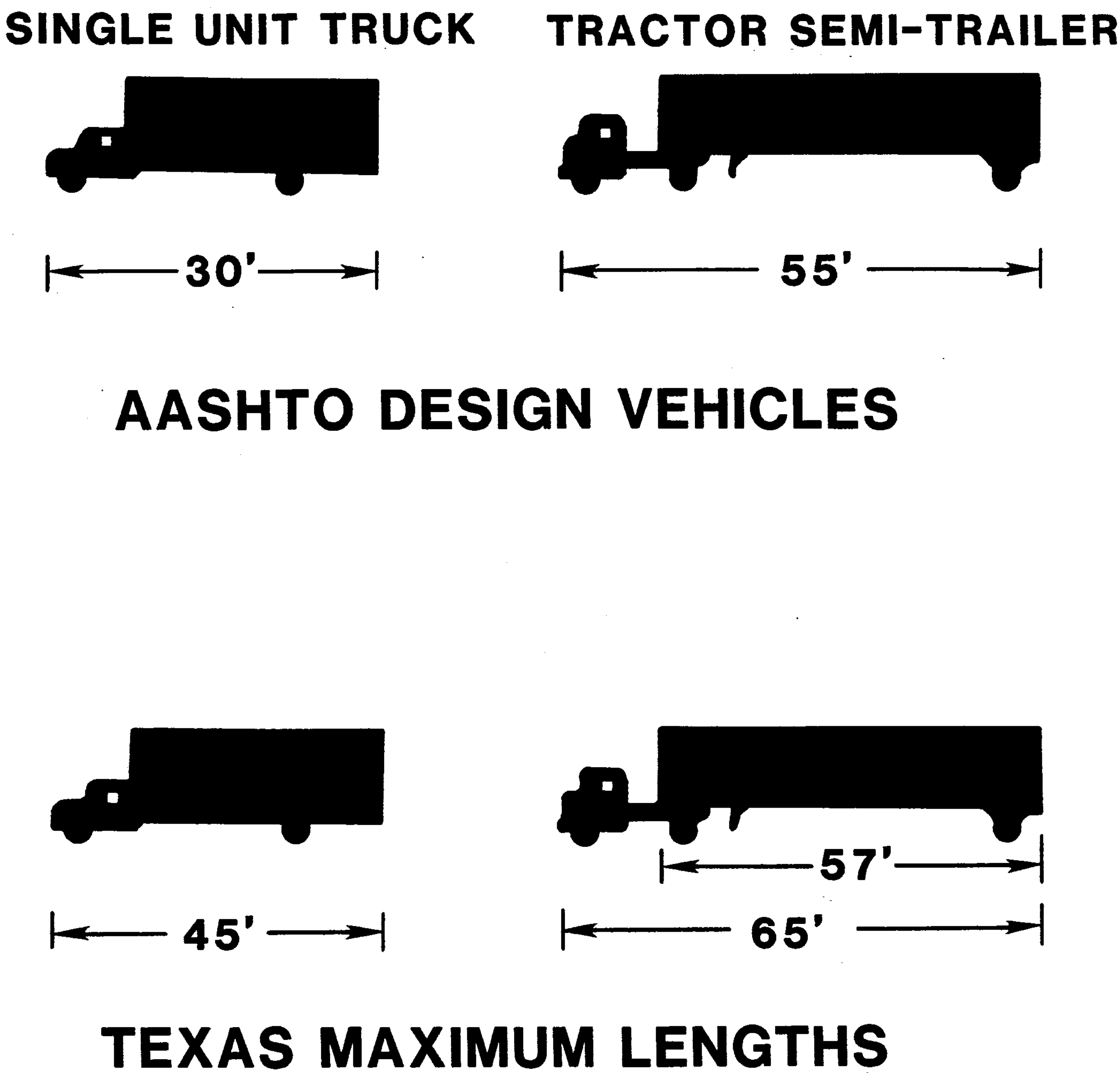


Figure 5. AASHTO Design Vehicles and Texas Maximum Lengths

Driver Eye Height

AASHTO provides a 3.5-ft driver eye height for design purposes; however, it reflects eye heights for passenger cars, not trucks. This low eye height value yields conservative estimates for truck driver sight distances and thus is assumed to provide a factor of safety in stopping distance calculations.

Gordon (8) gives the following values for driver eye height for various truck configurations:

CAB STYLE	DRIVER EYE HEIGHT (ft)
cab-under-engine	3.08
cab-over-engine	7.84
cab-behind-engine	8.41

These values for the cab-over-engine and cab-behind-engine truck types were obtained from a study by the Urban Behavioral Research Associates entitled "Determination of Motor Vehicle Eye Height For Highway Design." The two different truck cab designs from three different truck manufacturers were used to determine eye height and field of vision. A total of six trucks were used - the makes being GMC, Ford, and Mack.

The cab-under configuration has a driver eye height value below that provided by AASHTO. The cab-under truck (9) design, shown in Figure 6, was designed by the Strick Corporation to increase the payload volume of large truck combinations by 16 to 38 percent over existing truck configurations. This concept was introduced in 1977 but has not "caught on," due most likely to adverse reaction by drivers from a safety standpoint. The driver eye height of this vehicle is approximately 3.08 ft, which would place the truck driver at a serious sight distance disadvantage on a vertical curve. The AASHTO assumption that increased eye heights afforded truck drivers compensate for inferior braking performance would not be applicable. Although this type of truck is not in use at this time, safety improvements may be developed that would make this vehicle type practical.

Middleton et al. (15) performed an evaluation of the potential for hazards to trucks operating on crest vertical curves designed to AASHTO standards. A portion of the analysis was devoted to the determination of truck driver eye heights to be used in the study. Truck types were divided into three categories: cab-over-engine (COE), conventional or cab-behind-engine (CONV), and low cab-over-engine (LCOE). Two sources of data were used to define the driver eye height for each cab type. The first data set was obtained from a study performed by Vector Enterprises in 1982 for the National Highway Traffic Safety Administration. Truck driver eye heights were obtained for Ford, General Motors, Mack, and Freightliner trucks. The second data set was provided by Mack, Freightliner, and International Harvester who were each asked to provide driver eye heights for their most common cab types.

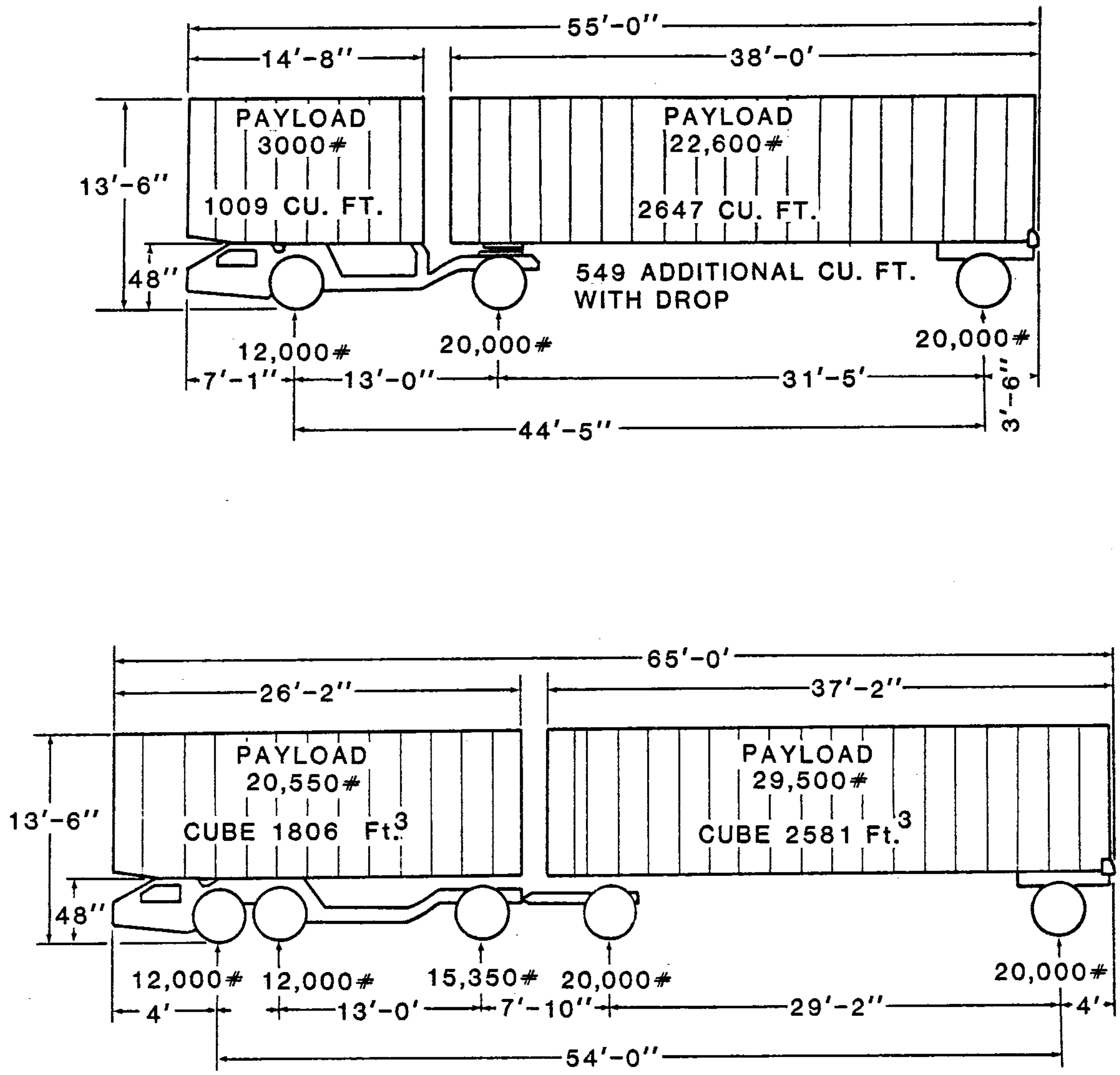


Figure 6. Cab-Under Truck Design (9).

The Mack data provided eye heights for seven cab types with drivers' seats in the "mid-ride" position. Ranges were provided to account for the differences due to tires, axles, and suspension.

The International Harvester Corporation provided eye height ranges for three cab configurations. The lower values of the ranges were based on a 5 percentile female driver eye height. The ranges also accounted for variations in heights due to tire size, suspensions, cab mounting, seat configurations, seat adjustments, etc.

Freightliner provided seat height information for two cab configurations. The seat heights were adjusted to their lowest positions and their heights above the ground were determined. The results of a study which provided a distribution of driver eye heights above the driver's seat were combined with the Freightliner data to obtain a range of driver eye heights.

The following values of driver eye height found by Middleton et al. for the three cab types are shown below:

CAB TYPE	DRIVER EYE HEIGHT, ft
CONV	7.75
COE	8.92
LCOE	7.58

This data seems to conflict with the study cited by Gordon in that the eye heights for the CONV and COE cab types are reversed. This illustrates the need for further research to determine of a representative value for truck driver eye heights.

Gordon states that an explicit procedure for determining eye-height standards would be desirable. This procedure should specify the vehicle model years to be sampled, the driving population and the proportion that must be accommodated, and the procedure by which measurements must be taken. Gordon suggests photographing unaware drivers seated in their natural positions within the vehicle.

Vehicle Headlight Height

Vehicle headlight height is important in calculating available sight distances in the design of sag vertical curves. AASHTO assumes a 2-ft headlight height and 1-degree upward divergence of the light beam from the longitudinal axis of the vehicle. The distance at which the light beam strikes the pavement is assumed to be the sight distance on the curve. Gordon (8) has found that the rise angle from the truck driver's eye to the top of the windshield permits the driver to see beyond the area lit by a 1-

degree rise angle from the headlights. Therefore no unusual visibility problems are encountered by trucks on sag vertical curves.

Weight-to-Horsepower Ratio

AASHTO defines the weight-to-horsepower ratio as the gross weight of the vehicle divided by the net engine horsepower. Net engine horsepower is the horsepower obtained at the flywheel, considering reductions in available horsepower due to accessories under the hood such as water pumps, air compressors, alternators, and fuel pumps. Reductions in horsepower due to driveline friction in the transmission and rear axle, or reductions in horsepower due to rolling resistance of the truck tires are not considered. Net horsepower is usually within 90 percent of the nameplate or gross horsepower of the engine.

AASHTO currently uses a weight-to-horsepower ratio of 300 to 1 in determining profile grades. A weight-to-horsepower ratio of 400 to 1 has been used in the past, the new value representing the improved performance of trucks over the past several decades. AASHTO warns, however, that in certain instances the 300 to 1 ratio may be inadequate.

Figure 7 (1) shows the historical trend in weight to horsepower ratios from 1949 to 1973. For a 80,000 pound vehicle, the 1949 average weight-to-horsepower ratio of about 550 had declined to 400 by 1973. This reduction is due to improved efficiencies in the engines and transmissions available in heavy trucks. Recent research findings indicate that there is general tendency for further weight/horsepower reduction.

Truck Braking Distances

Heavy vehicle braking performance is affected by many factors: tire type and condition, weight of the vehicle, road surface characteristics, the number of axles, and the number of tires per axle. Several studies have addressed the determination of heavy vehicle braking distance (4,8,10,11). Unfortunately, comparing one test with another must be done cautiously as each test was performed under unique conditions. Pavement friction, selection of drivers, condition of the vehicle, and study procedures each varied from test to test.

Figure 8 depicts the results of heavy vehicle braking studies conducted in Virginia (1969) and Alberta, Canada (1970) (4). These tests were conducted using trucks in excellent condition. Brakes were constantly checked by skilled mechanics. Tires and equipment were relatively new, and drivers were carefully picked. Although these values represent optimum conditions, deterioration of any of the above factors is known to occur in actual operation. The AASHTO braking curve represents braking distances presented on page 136 of the 1965 Bluebook. These tests found trucks operating in "optimum" conditions closely reflect braking distances used by AASHTO in design. However, any deterioration in equipment will result in braking distances above the minimum distances estimated using the AASHTO Bluebook.

Truck braking tests were performed in Utah to determine the braking performances of single, double, and triple combination trucks (10). The tests were performed on both wet and dry pavement surfaces. The wet and dry

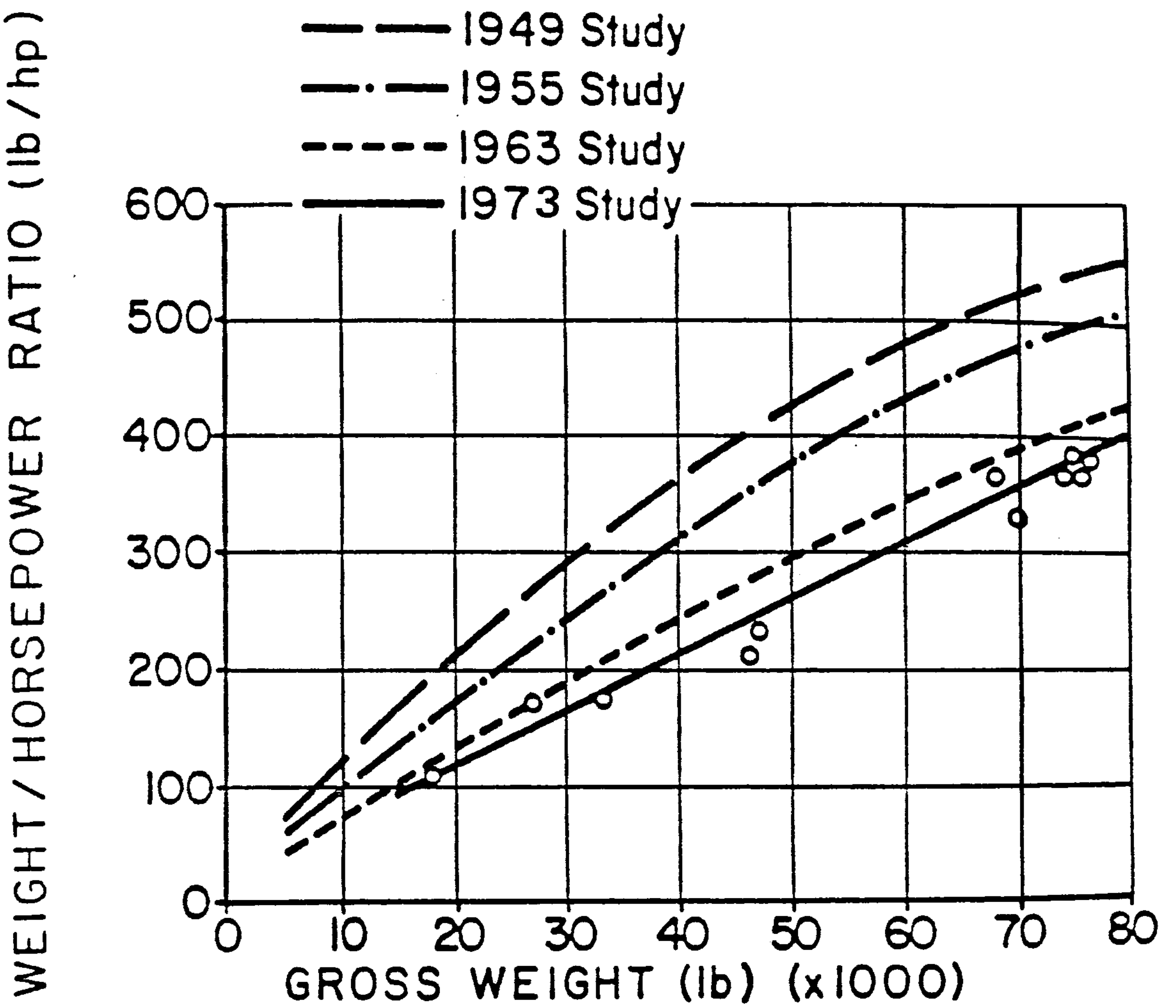
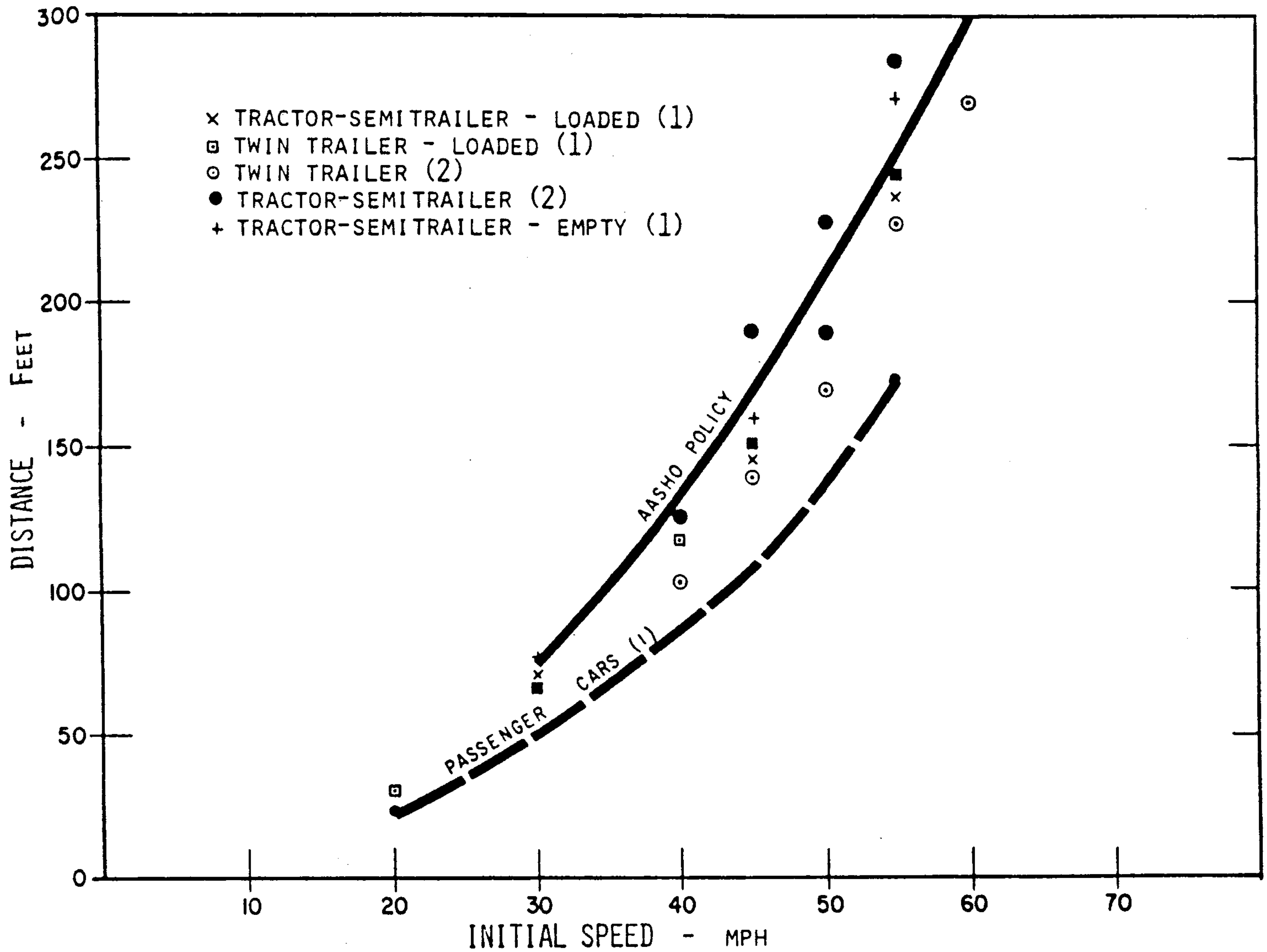


Figure 7. Historical Trend in Weight to Horsepower Ratios from 1949 to 1973 (1).



SOURCE:

(1) "The operational and safety characteristics of twin trailer combinations." Virginia Twin Trailer Commission Report, May 1969.

(2) "Twin trailer combination in Alberta." Alberta Department of Highways and Transport, April 1970.

Figure 8. Heavy Vehicles Braking Studies in Virginia and Canada. (4).

pavement coefficients of friction were .64 and .92, respectively. The test was divided into two series. The first series involved the testing of fully loaded triple and double combinations at speeds of 20, 30, and 40 miles per hour. There were no wet pavement tests at 40 miles per hour and no double tests at 20 miles per hour. The first series of tests found there was little tendency for truck combinations to jackknife on the wet surface; triple combinations appeared more stable than the doubles. The braking systems on the double and triple units were designed so that the rearmost axles locked up before the axles in front.

The second series of tests replicated the first series, except that a fully loaded single was added. Tests were conducted on wet and dry surfaces; the single was not tested at 40 miles per hour on the wet surface. The researchers feared the truck might jackknife at this speed, since it had already shown this tendency at 30 miles per hour. The triples again exhibited the ability to stop in a straight line on wet pavement surfaces. Table 4 shows the results and axle loadings of both series of tests. A graphical representation of the results of both series of tests is illustrated in Figure 9.

Peterson points out that the U.S. DOT, FHWA "Motor Carrier Safety Regulations" (12) specify deceleration rates of 21 ft per second per second for passenger cars and 14 ft per second per second for truck combinations. As a result, a car should stop in two-thirds the distance required by a truck. DOT regulations also specify that a truck must stop within a distance of 40 ft from an initial velocity of 20 miles per hour. Based on the 40 ft stopping distance requirement and the 14 ft per second per second deceleration rate, the resulting required braking distance vs. initial speed has been plotted in Figure 9. Also shown are the stopping distances of passenger cars on wet and dry pavements with coefficients of friction of .64 and .9, respectively, as predicted by the AASHTO braking distance equation. Note that the DOT stopping distance curve is considerably higher than the AASHTO criteria. A truck traveling 30 miles per hour on dry pavement may require approximately 50 ft over and above the braking distance required by a passenger car traveling at the same speed on the same pavement.

Peterson found that the data from these tests supports the two-thirds stopping distance rule, or that cars can stop in two-thirds the distance required by a truck. He also cites the tests found in the "Report of the Twin Trailer Study Commission to the Governor and the General Assembly of Virginia" performed in the Commonwealth of Virginia as support of this rule.

The Utah tests showed that doubles require a slightly longer stopping distance than singles, and that triples require slightly longer stopping distances than doubles. The three truck combinations were performed within U.S. DOT specifications. On wet pavements, triples were more stable than doubles, and doubles were more stable than singles. There was no observable difference in stability on dry pavements.

Figure 10 compares braking results obtained in Utah with similar tests performed in California and Alberta (10). The figure shows braking distances observed from single, double, and triple combinations stopping from various speeds on wet pavements. The Utah data shown on this bar chart represent the data in Table 4 of this report. As noted earlier, the coefficients of friction measured at the Utah site were .64 and .92 for wet

STOPPING DISTANCE TESTS-SERIES 1
MAY 30, 1974

Speed	Pavement Condition	DOUBLE REACTION	DOUBLE BRAKING	DOUBLE TOTAL	TRIPLE REACTION	TRIPLE BRAKING	TRIPLE TOTAL
20	DRY						62.4
20	WET				11.8	39.2	51.0
30	DRY	39.3	60.5	99.8	74.0	69.0	143
30	WET	17.1	73.4	90.5	11.4	85.3	96.7
40	DRY	70.4	103.3	173.7	21.8	133.6	155.4
40							
40*	DRY				1.0	129.0	130.0

STOPPING DISTANCE TESTS-SERIES 2
JUNE 11, 1974

Planned Speed	Pavement Condition	Test Number	SINGLE ACTUAL MPH	SINGLE REAC-TION	SINGLE BRAK-ING	SINGLE TOTAL	DOUBLE ACTUAL MPH	DOUBLE REAC-TION	DOUBLE BRAK-ING	DOUBLE TOTAL	TRIPLE ACTUAL MPH	TRIPLE REAC-TION	TRIPLE BRAK-ING	TRIPLE TOTAL
20	DRY	1	16.0	13.6	23.5	37.1	18.5	15.2	21.0	36.2		10.4	24.9	35.3
20	DRY	2	17.0	9	17.9	26.9					21.0	14.5	27.1	41.6
20	WET	1	16.0	1.3	19.0	20.3	18.5	16.0	23.9	39.9				
20	WET													
30	DRY	1	28.0	11.1	45.8	56.9		22.9	64.3	87.2	27.0	13.5	54.8	68.3
30	DRY	2	29.0	9.7	58.9	68.6	30.0	24.1	50.6	74.7	29.0	13.7	60.2	73.9
30	DRY	3		11.4	54.6	66.0								
30	WET	1	25.0	6.6	55.1	61.7	28.5	17.0	85.0	102.0				
40	DRY	1	39.0	65	55.9	120.9	40.0	23.8	88.0	111.8	39.0	25.9	100.0	125.9
40	DRY	2	39.0	11.7	123.3	135.0	40.0	36.8	93.5	130.3	40.0	16.6	92.4	109.0
40	WET	1					38.1	31.0	83.8	114.8	38.0	27.6	112	139.6
40	WET	2									41.0	6.4	104.9	111.3

Table 4. Results of Truck Braking Tests (10).

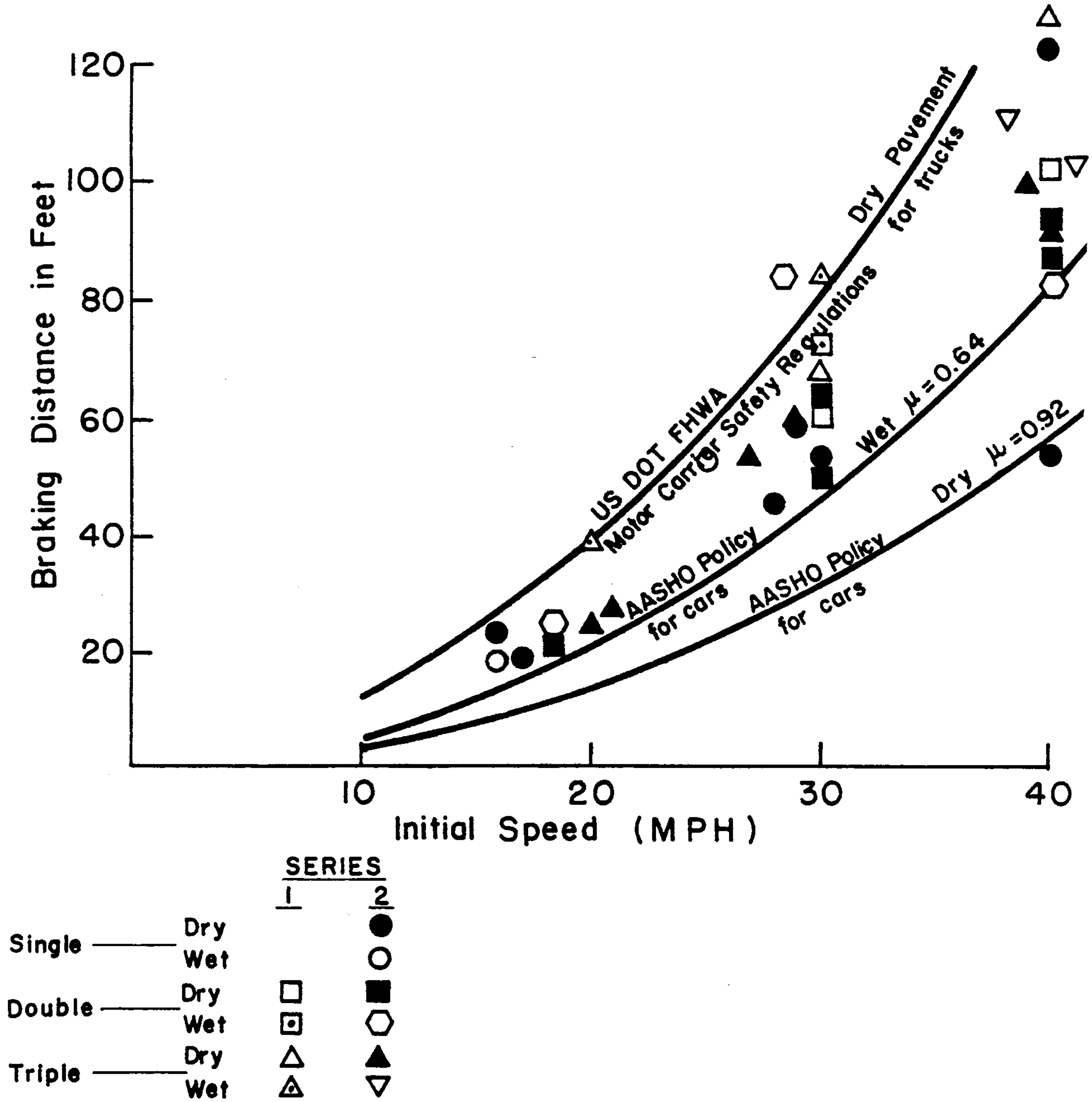


Figure 9. Graphical Representation of the Results of the Braking Tests (10).

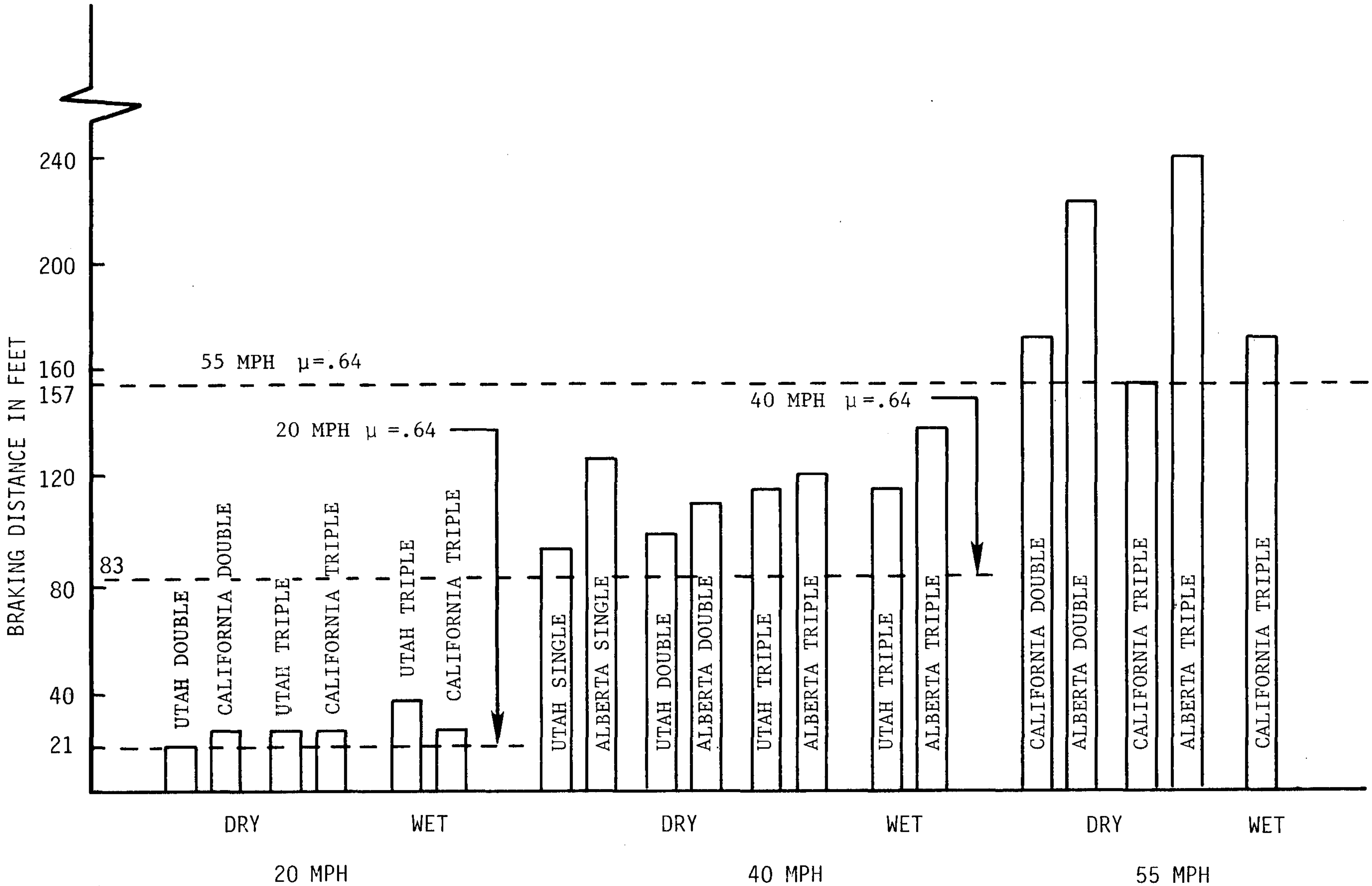


Figure 10. Braking Results in Utah Versus Similar Test in Califronia and Canada (10).

and dry pavements, respectively. The horizontal dashed lines on the chart represent braking distances predicted by the AASHTO braking distance equation:

$$\text{Braking Distance} = V^2/30f \tag{1}$$

Where:

$f = .64$

The Utah combinations exceed the AASHTO predicted values on wet pavement ($f = .64$) in every case. Even when the trucks were tested on dry pavements ($f = .92$), they exceeded braking distances predicted by AASHTO with a pavement coefficient of friction of .64. It is suspected that if the trucks were tested at coefficients of friction of .35 to .30, from 20 to 55 miles per hour, respectively (criteria used by AASHTO for stopping distance requirements), they would also exceed the values predicted by AASHTO. These observations suggest that the AASHTO stopping distance equation may need to be modified to more closely represent the braking distances of truck combinations.

Gordon (8) cites a study performed in 1974 by the Bureau of Motor Carrier Safety, which tested the braking performance of 1200 trucks of various types. The trucks were selected at vehicle weighing stations in California, Michigan, and Maryland. The tests measured the distance required to stop the vehicle from a speed of 20 miles per hour once the driver's foot touched the brake control. Gordon states that the results of the test bear a functional relationship to braking distances at other speeds and on other road surfaces. Figure 11 shows the cumulative percent of trucks stopping at or below the distance shown on the abscissa. Three types of trucks are represented in the figure: three-axle trucks; the 2-S2; and the twin-trailer combination. The three-axle truck is a single unit with three axles. The 2-S2 is a two-axle truck-tractor pulling a two-axle trailer. Gordon assumes that the braking performance of the 2-S2 may be taken as representative of 2-S1 and 3-S2 combinations. The twin-trailer combinations represent vehicles with total axles ranging from 5 to 11.

From the graph, the 50 percentile passenger car stopping distance is 21.75 ft. This corresponds to a deceleration rate of 19.87 ft per second per second. The average 50 percentile truck braking distance is 34.71 ft. This corresponds to a deceleration rate of 12.45 ft per second per second. The ratio of deceleration rates of trucks to cars is 12.45/19.87, or approximately two-thirds. The stopping distance ratio of cars to trucks is 21.75/34.71, which is also approximately equal to two-thirds. This is consistent with the relationship of passenger car stopping distances to truck stopping distances found in the Utah study and supports the two-thirds rule stated by Peterson in that report.

Truck braking distance depends on many variables. Several studies performed to determine the required braking distances of truck combinations indicate that the AASHTO braking equation is not adequate for todays larger and heavier trucks. The braking distances required by the Federal Motor Carrier Safety Regulations allow for truck stopping distances which exceed AASHTO's desirable design limits.

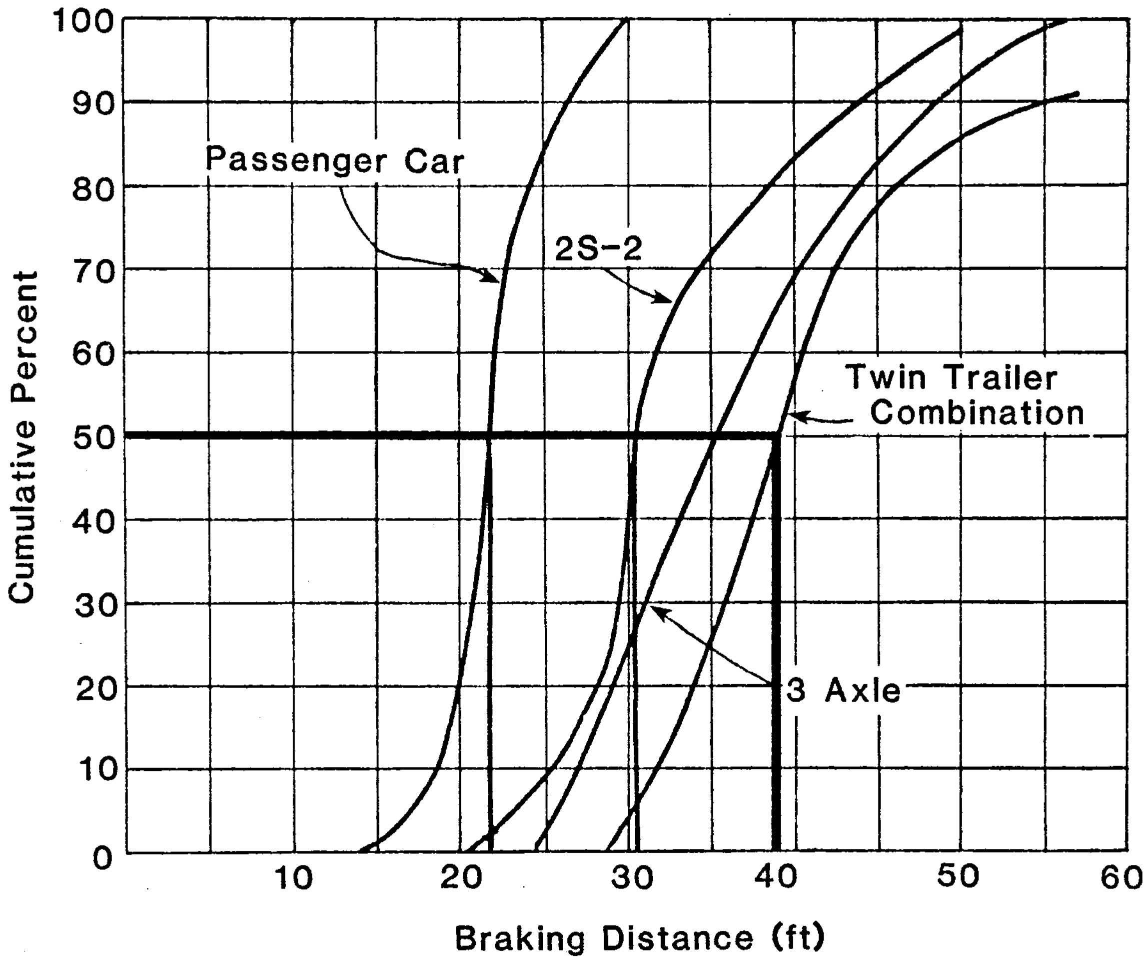


Figure 11. Cumulative Percent of Trucks Stopping at or Below the Distance Shown on the Abscissa (8).

ELEMENTS OF DESIGN

SIGHT DISTANCE

Perception-Reaction Time

Brake reaction time is defined by AASHTO (1) as the interval between the instant that the driver recognizes the existence of an object or hazard on the roadway ahead and the instant that the driver actually applies the brakes. It is commonly referred to as perception-reaction time. AASHTO uses 2.5 seconds as the perception-reaction time of most drivers on the highway for most conditions. For more complex intersections, or other areas where errors may occur by the driver, a decision sight distance should be provided. Decision sight distances require a longer reaction time and range from 10 to 14 seconds.

Some controversy exists regarding the 2.5 second perception-reaction time. Hooper and McGee (13) point out that the 60 percent higher braking distances of trucks over cars substantially reduce the perception-reaction component of the braking distance equation used by AASHTO. In addition, at higher design speeds, truck braking distance exceeds the total sight distance. Although Hooper and McGee report the results of many studies, they do not provide recommendations for an appropriate value of perception-reaction time.

In another publication, Hooper and McGee, in cooperation with Gordon (14), suggest that a 3.2 second perception-reaction time be used in braking distance determination. This value represents the 85th percentile perception-reaction time for the driving population. Table 5 shows the perception-reaction time for various percentiles of driving population based on their studies. The table breaks the perception-reaction time into its constituent elements. It should be pointed out that it is not certain that the summing of the elements in the table is a valid estimate of perception-reaction time, since it is thought that some of the elements are performed simultaneously.

Middleton et al. (15) suggest that truck drivers represent a more experienced portion of the driving population and may have different perception-reaction times than the driving population as a whole. A recommendation is made to investigate perception-reaction time for differences due to driver experience or other factors.

Braking Distance

AASHTO uses the equation:

$$d = V^2/30f$$

To determine required braking distances of passenger cars, where:

d = braking distance, ft;

V = initial speed, miles per hour; and

—Perception-brake reaction time for various percentiles of driving population (Stopping sight distance, sec)

Element	Percentile of drivers 50	75	85	90	95	99
Perception						
Latency	0.24	0.27	0.31	0.33	0.35	0.45
Eye movement	0.09	0.09	0.09	0.09	0.09	0.09
Fixation	0.20	0.20	0.20	0.20	0.20	0.20
Recognition	0.40	0.45	0.50	0.55	0.60	0.65
Decision	0.50	0.75	0.85	0.90	0.95	1.00
Brake reaction	0.85	1.11	1.24	1.42	1.63	2.16
Total	2.3	2.9	3.2	3.5	3.8	4.6

Table 5. Perception-Brake Reaction Times (14).

f = coefficient of friction between the tires and roadway.

It has been shown that cars stop in approximately two-thirds of the distance of heavy trucks. If d represents the stopping distance of heavy trucks, then 2/3d represents the braking distance of passenger cars. Since

$V^2/30f$ = braking distance of passenger cars

Then

$$2/3d = V^2/30f \tag{2}$$

And

$$d = 3V^2/60f \tag{3}$$

Reducing this equation further, we get

$$d = V^2/20f \tag{4}$$

This is the equation for truck braking distance proposed by Peterson (11).

No evidence was found in the literature to indicate any changes in the currently used "f" values in the AASHTO equation, and therefore these "f" values are assumed applicable. They represent wet pavements approaching their design lives. Since it has been found that drivers do not slow down on wet pavements, the design speed of the highway is used in calculating desirable stopping sight distance (1).

Combining the perception-reaction time and the braking equation results in the following equation for stopping-sight distance (SSD):

$$SSD = 1.47(V)T + V^2/20f \tag{5}$$

Where:

V = vehicle speed, miles per hour;

T = perception-reaction time, sec; and

f = coefficient of friction.

Table 6 shows the stopping sight distances calculated from the above equation, using perception-reaction times of 2.5 and 3.2. Also included for comparison are SSD values for passenger cars.

DESIGN SPEED (MPH)	REACTION TIME (SEC)	1.47V(T) REACTION DISTANCE (FT)	COEFFICIENT OF FRICTION (f)	ROUNDED STOPPING SIGHT DISTANCE (FT)		
				CAR*	TRUCK**	TRUCK***
20	2.5	73.5	0.4	107	144	124
25	2.5	91.88	0.38	147	200	174
30	2.5	110.25	0.35	196	270	239
35	2.5	128.63	0.34	249	345	309
40	2.5	147.00	0.32	314	438	397
45	2.5	165.38	0.31	383	538	492
50	2.5	183.75	0.30	462	652	600
55	2.5	202.13	0.30	538	763	706
60	2.5	220.50	0.29	634	903	841
65	2.5	238.88	0.29	725	1034	967
70	2.5	257.25	0.28	841	1204	1132

*$1.47V(T) + V^2/30f$ (CURRENT AASHTO DESIGN CRITERIA)

**$1.47V(T) + V^2/20f$ WHERE REACTION TIME, t = 3.2 SEC

***$1.47V(T) + V^2/20f$ (SEE ELEMENTS OF DESIGN - PERCEPTION REACTION TIME)

Table 6. Stopping Sight Distances

Effects of Grade on Braking Distance

AASHTO compensates for changes in braking distances of passenger cars due to grades by inserting the g-term in the denominator of the braking distance equation. The g-term represents the percent grade divided by 100 and is added to the "f" factor. Nothing was found in the literature to indicate that this method of compensation would not apply to truck braking performance. Thus, the formula for predicting truck braking performance on grades is:

$$d = v^2/20\ (f \pm g) \tag{6}$$

Table 7 shows the effect of grade on braking distances for heavy vehicles. On downgrades and upgrades of 3, 6, and 9 percent, the correction in braking distance is given for each design speed. These corrections represent the numerical differences between braking distance on level grade and braking distances on vertical grade. For instance, if a 9-percent downgrade exists on a facility with a 30-mph design speed, 45 ft are added to the braking distances calculated on level grade. The average running speed is used by AASHTO for the upgrade design speeds.

Decision Sight Distance

AASHTO recommends increased perception-reaction time when drivers are faced with complex or instantaneous decisions, when information is difficult to perceive, or when unusual maneuvers are required. In these instances, longer sight distance should be provided through the use of decision-sight distance. This decision-sight distance allows for the driver to detect an unexpected or difficult to perceive information source, recognize the nature of the hazard, select an appropriate speed or path, and initiate the maneuver safely and efficiently.

AASHTO states that decision-sight distance should be used at intersections, interchanges, locations where unusual or unexpected maneuvers are required, changes in cross section, and in areas where sources of information such as roadway elements, traffic control devices, and advertising compete for the driver's attention.

Due to the decreased maneuverability and increased stopping distances of trucks as compared to passenger cars, consideration should be given to the use of decision sight distance in the design of exclusive truck facilities. The decision-sight distances provided by AASHTO shown in Table 8 represent the driving population as a whole.

Passing Sight Distance

AASHTO design policy establishes minimum passing sight distances for two-lane highways. These distances were derived from operational characteristics of passenger cars and are not directly applicable to truck facilities' design.

If truck lanes are simply added to the existing cross section of multi-lane highways, modification for passing sight distance would be unlikely. However, if the facilities are to be placed in areas of restricted right-of-way, such as a freeway median, it may be necessary to restrict the cross

INCREASE FOR DOWNGRADES				DECREASE FOR UPGRADES			
DESIGN SPEED (MPH)	CORRECTION IN BRAKING DISTANCE (FT)			DESIGN SPEED* (MPH)	CORRECTION IN BRAKING DISTANCE (FT)		
	3%	6%	9%		3%	6%	9%
30	12	27	45	28	9	16	23
40	26	58	98	36	18	32	44
50	47	104	178	44	30	54	75
60	72	162	279	52	43	80	110
65	84	190	328	55	49	90	124
70	105	239	415	58	58	106	146

$$d = \frac{V^2}{20} (f \pm g)$$

*AVERAGE RUNNING SPEED IS USED BY AASHTO FOR UPGRADE DESIGN SPEEDS

Table 7. Effect of Grade on Braking Distances

Design Speed (mph)	Time(s)			Decision Sight Distance (ft)		
	Premaneuver					
	Detection & Recognition	Decision & Response Initiation	Maneuver (Lane Change)	Summation	Computed	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

Table 8. AASHTO Decision Sight Distances (1).

section to two lanes -- one in each direction, separated by a median barrier.

In this case, passing lanes should be provided intermittently as costs and operations permit. These passing lanes should be similar to auxiliary truck climbing lanes, the beginning and ending location being determined by the slower trucks occupying the right-most lane, allowing the faster moving vehicles to pass in the left-hand lane.

Since it would not be practical for these auxiliary passing lanes to be continuous, it is important that they be long enough to allow the faster truck to safely pass slower units. The required length of passing lane is dependent on the length of the passing vehicle, length of the vehicle being passed, and the running speed of each vehicle.

The time required for a vehicle to pass another is given by (4):

$$T_p = (L_f + L_s + 150)/1.47dV \tag{7}$$

Where:

L_f = length of faster vehicle, ft;

L_s = length of slower vehicle, ft;

150 = 75 ft for pullout and return maneuvers;

1.47 = conversion factor from miles per hour to ft per second and;

dV = speed differential between vehicles, miles per hour.

This equation is conservative, because it assumes a constant speed for the passing vehicle throughout the maneuver. Any acceleration by the passing vehicle during the maneuver will reduce the required passing time. The 150-ft dimension for pullout and return is not fully documented and appears questionable.

The distance in feet required by the passing vehicle to complete the entire maneuver (from beginning to end) is given as:

$$d = 1.47VT_p \tag{8}$$

Combining equations (8) and (9):

$$d = 1.47V(L_f + L_s + 150)/(1.47dV) \tag{9}$$

Or

$$d = V(L_f + L_s + 150)/dV \qquad (10)$$

In Texas, the current maximum legal length for truck combinations is 65 ft. Using this value in Equation 11 gives the passing distance required for one 65 ft combination to pass another. Assuming a 60 mile per hour running speed and the 150-ft pullout and return distance, the equation is reduced to the following form:

$$d = 16800/dV \qquad (11)$$

This equation represents passing distances on level grades. It can be modified to accommodate any combination of vehicle lengths and speed differentials. Figure 12 provides the required passing lane lengths as a function of the difference in speeds between the faster and slower vehicle. The calculation assumes both vehicles are 65 ft in length.

HORIZONTAL ALIGNMENT

Friction Factor

AASHTO uses the equation:

$$e + f = V^2/15R \qquad (12)$$

Where:

V = vehicle speed, miles per hour;

e = superelevation rate, ft per ft;

f = limiting side friction factor (coefficient of friction); and

R = radius of curvature, ft;

to determine minimum curve radii for a given highway design speed. For a given design speed, practical superelevation rate, and limiting side friction factor, a minimum curve radius is calculated.

This equation was derived from studies of passenger car operations. The "f" parameter (referred to as the side friction factor) is the maximum value which will result in a centrifugal force uncomfortable to the driver. Weinberg and Tharp (16) have expressed the concern that the maximum f values used by AASHTO fail to take into account the overturning tendency of a vehicle on a turn. A side friction factor which has not exceeded the "driver comfort" range may be of sufficient magnitude to cause a heavily loaded vehicle with a high center of gravity (CG) to overturn while

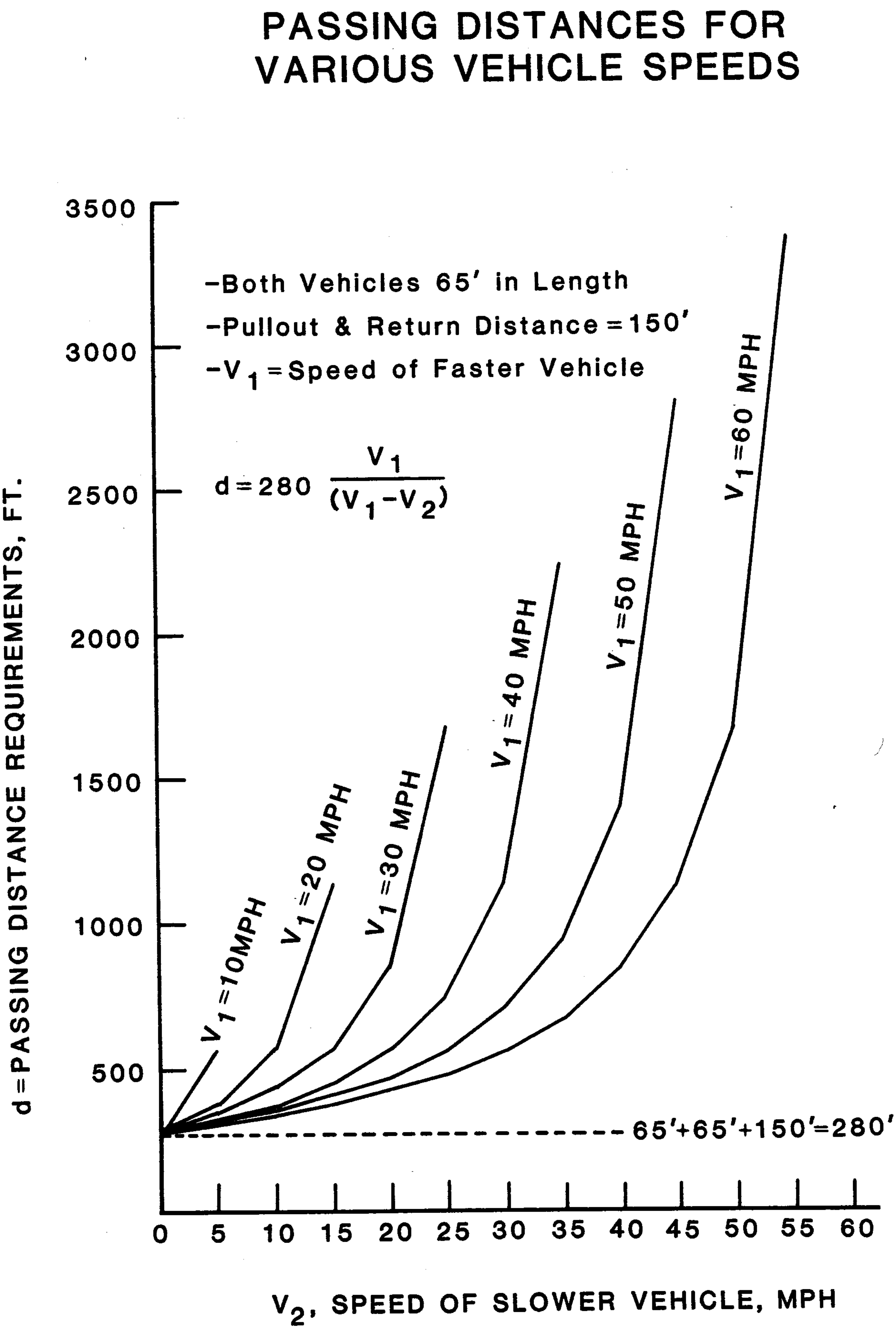


Figure 12. Passing Distances for Various Vehicle Speeds

negotiating a turn (4). The mechanics of the rollover process of heavily loaded truck combinations has been analyzed mathematically (17). Rollover thresholds of trucks have been established based on truck axle load, gross weight, width variations, and height of the payload center of gravity. The rollover threshold is defined as the maximum value of lateral acceleration which the vehicle can tolerate without rolling over.

Figure 13 shows the rollover threshold for various truck types and axle loadings for two conditions, a payload CG height of 105 inches, and a payload CG that varies with gross weight. The gross vehicle weights vary from 30,000 pounds to 88,000 pounds. It was assumed that the cargo was of a homogeneous weight of 34 pounds per cubic ft. When a load variation was considered (such as a half-loaded truck), it was assumed that materials were removed form the top of the load, leaving the remaining cargo evenly distributed along the floor of the cargo area. The 105-inch CG height corresponds to an 88,000 pound gross vehicle weight; a CG height of 83.5 inches corresponds to an 80,000 pound gross vehicle weight. The centers of gravity were measured with respect to the ground. Figure 13 indicates that rollover thresholds vary from .25 to .4 g's, where 1 g equals the weight of the truck.

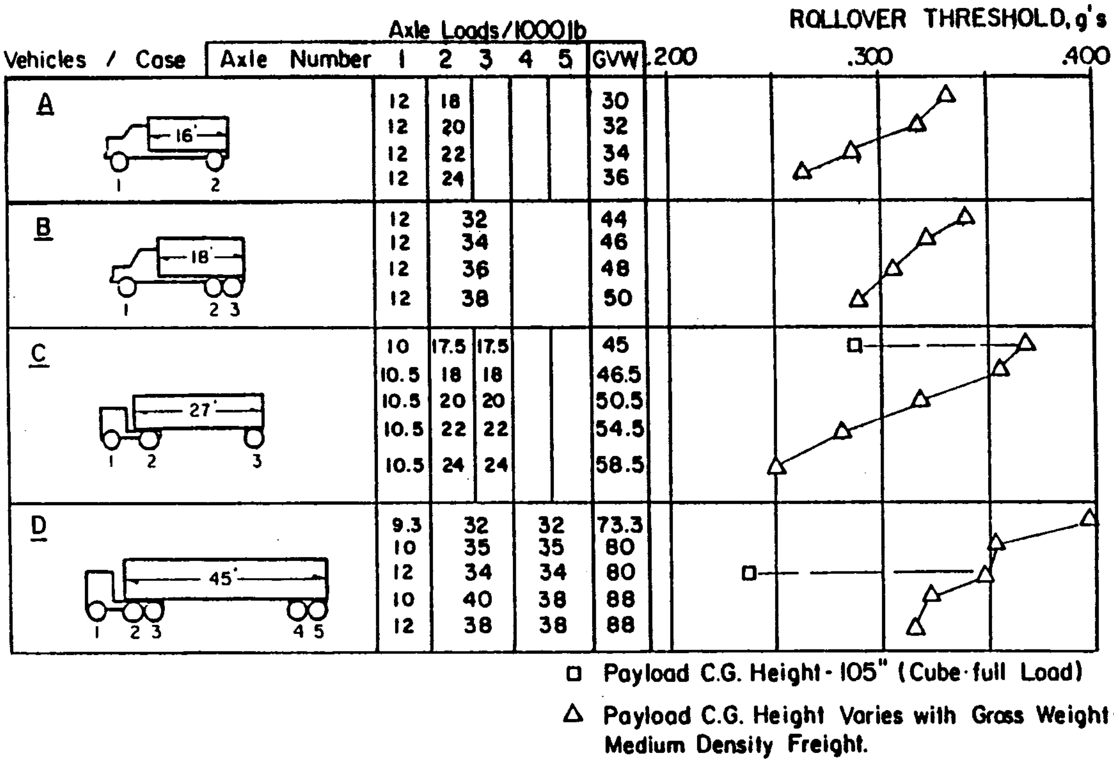
The influence of gross weight variations on the rollover threshold is illustrated in Figure 14. The rollover threshold of the truck combinations decreases as gross vehicle weight increases. Therefore, when trucks carry cargo of a homogeneous weight, the stability decreases as the gross weight of the truck increases. The rollover threshold in this case also varies from about .25 g's to .4 g's.

Figure 15 shows the influence of truck width on the rollover threshold. The center of gravity is assumed to be 83.5 inches above ground level. For truck bed widths varying from 96 inches to 108 inches and tire widths from 96 inches to 108 inches, the rollover threshold varies from, again, .25 g's to .4 g's.

Figure 16 shows the influence of payload CG height on the rollover threshold. From this figure, the rollover threshold varies from .22 to .45 g's for an 80,000 pound vehicle.

The loading characteristics, axle weights, and track widths shown in the previous figures are typical of the trucks operating on the road today. It can therefore be concluded that the threshold of rollover of a typical truck may be in the area of .25 g's. The maximum lateral force developed by the tires on the roadway must be less than about one-quarter of the weight of the truck to avoid a rollover condition.

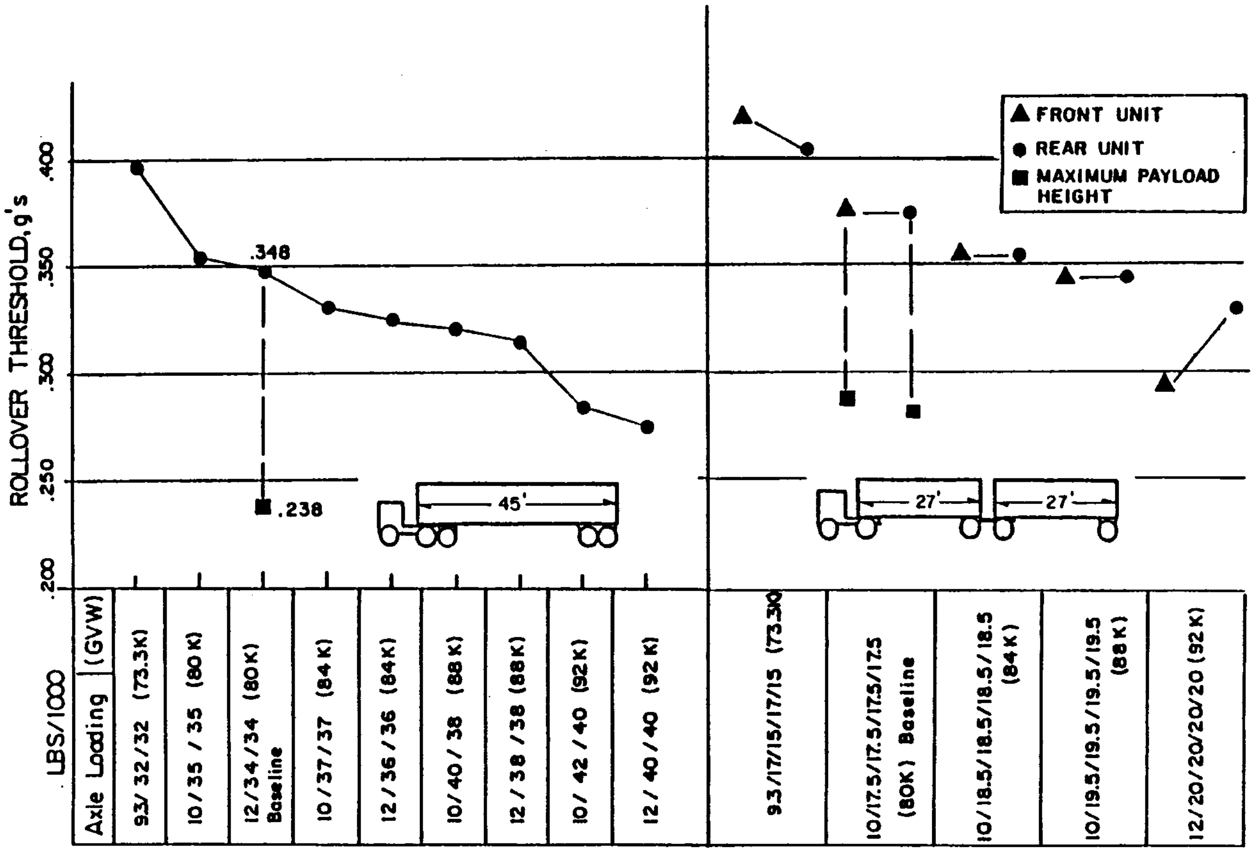
The side friction factor used by AASHTO is a coefficient of friction value. The coefficient of friction is found by dividing the friction force developed on the pavement by the normal force, which is the weight of the vehicle. Similarly, the g forces used above are a ratio of the lateral forces developed by the truck on the pavement and the weight of the truck. The coefficient of friction of a pavement can be thought of as the maximum lateral g forces which can be developed by a vehicle on a turn. If the coefficient of friction on a curve is .3, then a vehicle rounding the curve can develop a maximum of .3 g's on the turn. If the rollover threshold is



Vehicles / Case	Axle Loads/1000 lb, Axle Number 1	2	3	4	5	GVW
A	12	18				30
	12	20				32
	12	22				34
	12	24				36
B	12	32				44
	12	34				46
	12	36				48
	12	38				50
C	10	17.5	17.5			45
	10.5	18	18			46.5
	10.5	20	20			50.5
	10.5	22	22			54.5
	10.5	24	24			58.5
D	9.3	32		32		73.3
	10	35		35		80
	12	34		34		80
	10	40		38		88
	12	38		38		88

Influence of axle load variations on rollover threshold.

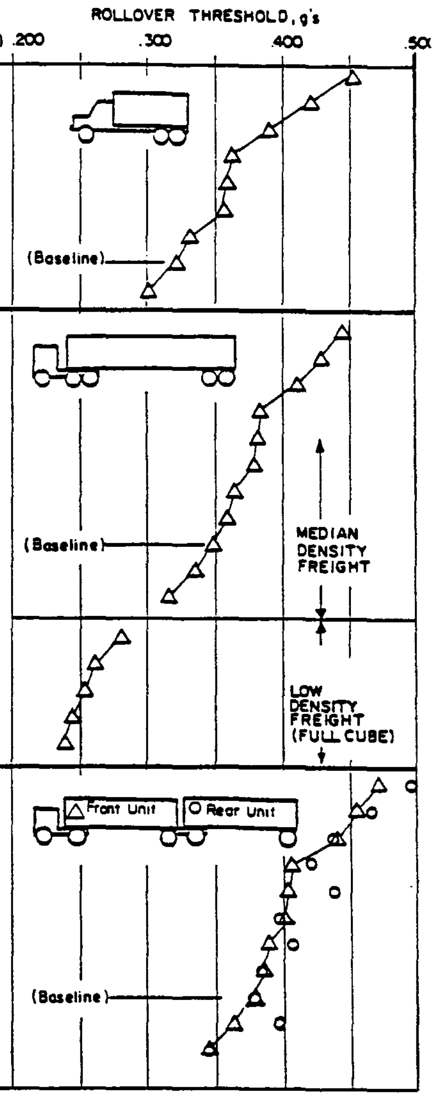
Figure 13. Rollover Threshold Values (17).



Influence of gross weight variations on rollover threshold.

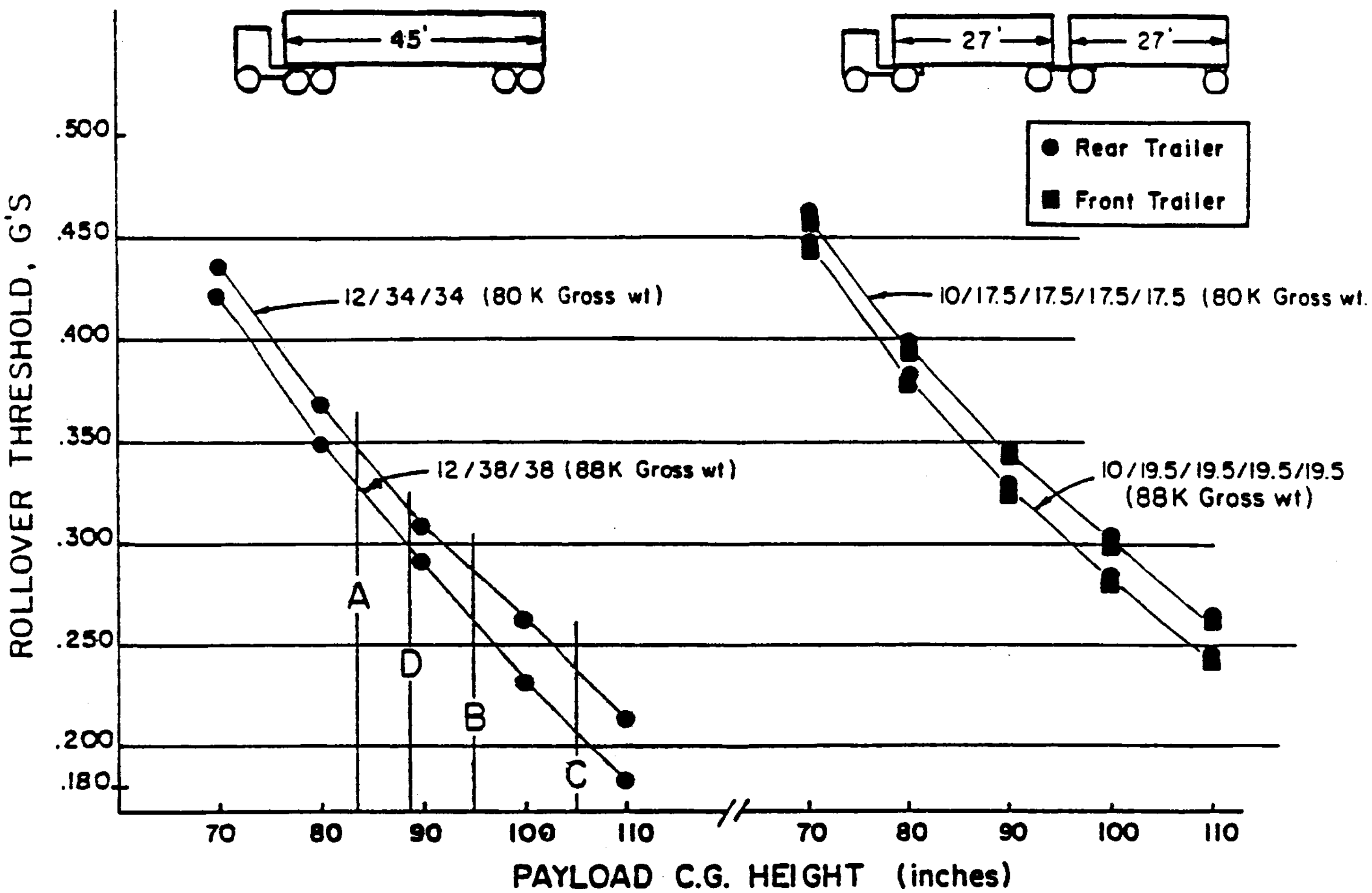
Figure 14. Influence of Gross Weight Variations on Rollover Threshold (17).

Case	Truck or Trailer Dim. Bed Width	Spring Spacing	Width Across Tires	Tractor Width Across Tires (in)	Gross Vehicle Weight (lb)
1	108	50	108		46 K
2	105	47	105		46K
3	102	44	102		46K
4	102	38	102		46K
5	102	44	102		50K
6	99	41	99		46K
7	102	38	96		46K
8	96	38	96		46K
9	102	38	96		50K
1	108	50	108	102	80K
2	105	47	105	102	80K
3	102	44	102	102	80K
4	102	44	102	102	88K
5	102	38	102	96	80K
6	102	44	102	96	80K
7	99	41	99	96	80K
8	102	38	96	96	80K
9	96	38	96	96	80K
10	102	44	102	96	88K
11	102	38	96	96	88K
12	102	44	102	102	80K
13	102	44	102	96	80K
14	102	38	102	96	80K
15	96	38	96	96	77K
16	102	38	96	96	80K
1	108	50	108	102	80K
2	105	47	105	102	80K
3	102	44	102	102	80K
4	102	38	102	96	80K
5	102	44	102	96	80K
6	102	44	102	102	88K
7	99	41	99	96	80K
8	102	38	96	96	80K
9	96	38	96	96	80K
10	102	44	102	96	88K
11	102	38	96	96	88K



Influence of width variations on rollover threshold.

Figure 15. Influence of Width Variations on Rollover Threshold (17).



Influence of payload c.g. height on rollover threshold.

Figure 16. Influence of Payload Center of Gravity Height on Rollover Threshold (17).

above .3 g's, then the vehicle will skid; if is does not, the vehicle will roll over.

To avoid vehicle rollover, the coefficient of friction needed on a curve should be below the rollover threshold of the vehicle. This is not to be taken to mean that highway curves should not provide as much friction as possible. It indicates that the degree of curvature, superelevation, and/or the design speed of the curve should be of such values as not to cause the vehicle to develop lateral friction forces exceeding the rollover threshold of the vehicle.

For rural highways and high speed urban streets, AASHTO's f values are .17 or below. This is well under the .25 limiting value for trucks. Thus, no problems should be encountered in the application of AASHTO curve design standards to exclusive truck facilities.

Results of a computer analyses and experimental measurements by MacAdams et al. show that modest differences do exist in wheel-to-wheel friction factor values on most vehicles during steady turning conditions. The two primary sources of friction factor variations are (32): 1) geometric properties of vehicles, and 2) normal driver steering fluctuations during curve negotiation. Consequently, even when drivers are capable of steering a curve in an ideal manner without mild steering oscillations, wheel-to-wheel friction factor differences would still exist because of the basic vehicle characteristics which set it apart from a "point-mass" object.

Despite the presence of these wheel-to-wheel friction factor variations, no evidence was found to indicate that the observed friction factor variations would lead to significantly reduced stability margins. Even if the available tire/road friction level was reduced to a value below the demand of the tire having the greatest friction requirement, no vehicle instability would occur. Interestingly, the minimum level of the tire/road friction identified for maintaining stability of passenger cars was found to be equal to the "point mass" design values for the curve. However, the minimum level of friction necessary for maintaining stability of the five axle tractor-semitrailer was about 10 percent higher than the point-mass design value.

The safety margins provided by AASHTO design are generous for both types of vehicles. For example, and AASHTO curve with a design speed of 70 mph (f_{max} = 0.10) and an assumed wet road friction level of 0.30, the margin of safety would be 0.20 g's for the passenger car (0.30 - 0.10) and 19 g's for the tractor-semitrailer (0.30 - 0.11). Note, this conclusion applies to negotiation of horizontal curves well after the PC and does not apply to transition section or "overshoot" behavior caused by transitions at the start of a horizontal curve (32).

Increased margins of safety may appear necessary for high center of gravity vehicles operating on lower design speed curves. However, the existing AASHTO policy still provides ample means for reducing side friction demand while also retaining near maximum superelevation rates.

Hydroplaning

The myth that trucks do not hydroplane began after NASA studied hydroplaning on aircraft tires. Because aircraft tires are unaffected by a wide variation of tires loads, NASA found that hydroplaning speeds of aircrafts were a simple function of tire pressure. The speeds examined during aircraft hydroplaning were above that which could be achieved by highway vehicles. Because of the higher speeds, and the similar pressure between truck and aircraft tires, the myth that trucks do not hydroplane developed.

The original equation developed by NASA (18):

$$V_p = 0.592 \left(\frac{F_V \, 288}{A_G \, \zeta C_L}\right)^{0.5} \tag{13}$$

Where:

V_p = minimum tire hydroplaning speed on flooded pavement, mph;

F_V = vertical force of tire, lbs;

A_G = gross area, in^2;

ζ = mass density of water, slugs/ft^3; and

C_L = tire footprint lift coefficient, unitless.

This equation can be simplified by assuming (18):

a) ζ = 1.94 slugs/ft^3

b) $p = 10.35 \sqrt{p}$

c) $C_L = 0.642$

Substituting the above assumptions:

$$V_p = 10.35 \sqrt{p} \tag{14}$$

Tests were performed at the Texas Transportation Institute (TTI) to determine the reason why so many unloaded tractor-trailers lose control during wet whether. During the tests, TTI observed hydroplaning when the tractor-trailer reaches a speed of 58 mph with tire inflation pressure of 75 psi. Table 9 (18) illustrates that all loaded tractor-trailer tires have high minimum hydroplaning speeds. For unloaded conditions, large differences in hydroplaning speeds exist between the steering axle and driving axle tires of the tractor. These large differences imply that there are large differences in wet cornering traction as well. A large differential traction between steering axle and driving axle tires when the tractor enters, transits, and exits a water puddle can create a rotational movement about the tractor's center of gravity, causing the tractor-trailer to jackknife.

TRACTOR-TRAILER (IB WHEELS) TIRE FOOTPRINT CHARACTERISTICS. INFLATION PRESSURE = 100 LB/IN^2						
TIRE LOCATION	TRAILER LOAD CONDITION	LOAD PER TIRE ON AXLE, LB	TIRE FOOTPRINT			V_p mph
			WIDTH, IN.	LENGTH, IN.	ASPECT RATIO	
TRUCK FRONT (STEERING) AXLE	LOADED EMPTY	5720 4270	7.1 6.95	9.36 8.4	0.76 0.83	91.2 87.3
TRUCK FOR'D DRIVE AXLE	LOADED EMPTY	4285 1285	7.24 6.84	8.22 4.34	0.88 1.58	84.7 63.2
TRUCK REAR DRIVE AXLE	LOADED EMPTY	3825 1120	7.33 6.76	7.03 3.88	1.04 1.74	78.0 60.3
TRAILER FORWARD AXLE	LOADED EMPTY	4275 1085	7.47 4.83	8.0 4.97	0.93 0.97	82.4 80.7
TRAILER REAR AXLE	LOADED EMPTY	3970 840	7.28 5.03	8.42 4.94	0.86 1.02	85.7 78.7
$V_p = 7.95\sqrt{p(w/l)^{-1}}$ mph						

Table 9. Tractor-Trailer Tire Footprint Characteristics (18).

Based on field tests, TTI found that dynamic hydroplaning speeds of highway vehicle tires vary with both tire pressure and the tire footprint aspect ratio. The tire aspect ratio is the ratio of the surface contact zone width to the length.

In the original aircraft equation for hydroplaning, the aircraft tire footprint aspect ratio is a constant over a wide range of vertical tire deflections. If the aspect ratio is constant, then the lift coefficient C is constant, as in the original equation. For highway vehicles, the tire footprint lift coefficient is directly proportional to the tire footprint ratio. This is the modification that is needed in the aircraft equation:

$$C_L = K(w/l) \tag{15}$$

substituting $C_L = 0.642$ (same as the original equation)

and $(w/l) = 0.59$ into the above equation and solving for K.

Substituting $C_L = 1.089(w/l)$ into the original equation yields:

$$V_p = 7.95\sqrt{p(w/l)^{-1}} \tag{16}$$

Where:

p = inflation pressure in lb/in; and

(w/l) = tire footprint aspect ratio.

The new minimum dynamic hydroplaning equation seems to provide reasonable speed for all types of pneumatic tires. Further research is still needed to establish the accuracy of this equation.

Pavement Widening on Curves

If the design of EFT facilities is to be based on current legal sizes of trucks in Texas, then the current AASHTO policy on pavement widening on curves should be applicable. This is true since the AASHTO design vehicles have dimensions which are representative of current legal limits in Texas.

However, if the truck facilities are designed on the premise that 105-ft double and triple combinations are to be accommodated on the facilities at some time in the future, then AASHTO policy is not entirely adequate.

AASHTO recommends pavement widening on curves to make operating conditions on curves comparable to tangent sections. Pavement widening is needed because: (1) the truck occupies a greater width on curves since the back wheels track inside the front (this is known as "off-tracking"), and (2) drivers experience difficulty in steering their vehicles in the center of the lane.

The following formula gives maximum off-tracking values which predict accurately the actual measured values of off-tracking (4,10,11):

$$MOT = R1 * (R1^2 - L1^2 + L2^2 + L3^2 + L4^2 + L5^2)^{.5} \tag{17}$$

Where:

MOT = Maximum offtracking, ft;

R1 = Turning radius of outside front wheel, ft;

L1 = Wheelbase of tractor, ft;

L2 = Wheelbase of first trailer or semitrailer, ft;

L3 = Distance between rear axle and articulation point, ft;

L4 = Distance between articulation point and front axle of next trailer, ft; and

L5 = Wheelbase of trailer, ft.

The extra width required due to off-tracking can be computed from the formulas shown in Figure 17. The total extra width required is the sum of the off-tracking widths and the widths calculated using the equations of Figure 17.

Walton and Gericke (11) have computed the extra width requirements for two-lane roads to accommodate 105-ft double and triple combinations. The results are shown in Table 10.

Figures 18 and 19 show the off-tracking characteristics of a triple (2-S1-2-2), double (3-S2-4), and single combination truck on a 100-ft and 147.5-ft radius turns with maximum vehicle dimensions of 105 ft in length (11).

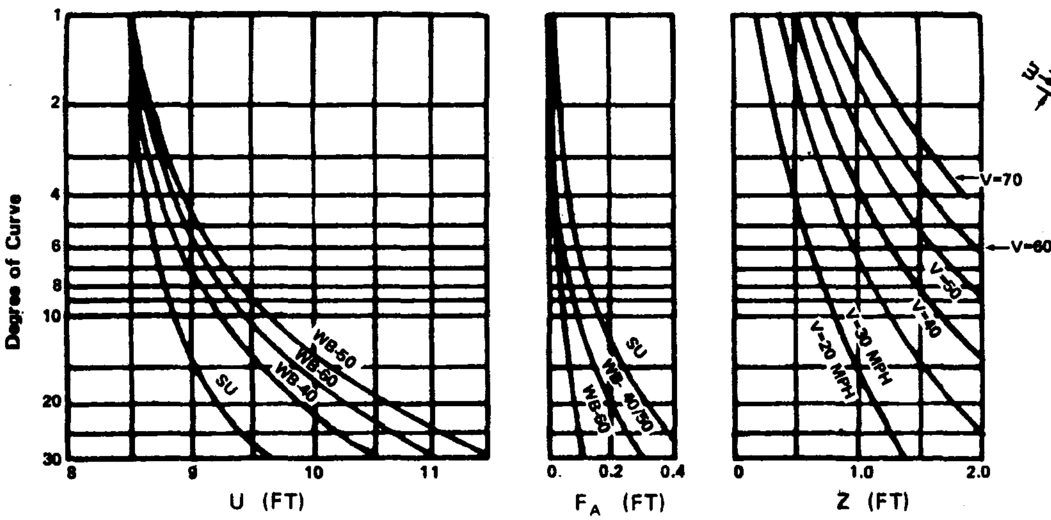
Sight Distance on Horizontal Curves

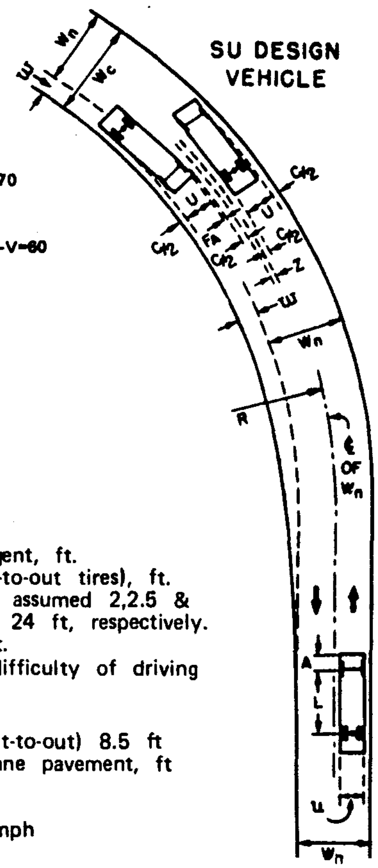
Adequate sight distance across the inside of horizontal curves is essential for providing safe stopping distance. Some common obstructions along the inside of a horizontal curve are walls, cut slopes, buildings, and guardrails.

The sight distance for horizontal curves is the chord at the center of the travel lane of the curve; the stopping distance is measured along the centerline of the inside lane around the curve. The sight distance values used are the values of stopping sight distance presented earlier in Table 6.

The design chart shown in Figure 20 shows the required value of the middle ordinate, M, which will satisfy the stopping distance requirements for trucks. It is obvious that the middle ordinate value is quite high when

Elements of Pavement Widening





(1) $w = W_C - W_n$

(2) $W_C = N(U+C) + (N-1)F_A + Z$

N = Number of lanes
w = widening for pavement on curve, ft.
W_C = width of pavement on curve, ft.

(3) $U = u + R - \sqrt{R^2 - L^2}$

(4) $F_A = \sqrt{R^2 + A(2L + A)} - R$

(5) $Z = V/\sqrt{R}$

W_n = width of pavement on tangent, ft.
U = track width of vehicle (out-to-out tires), ft.
C = lateral clearance per vehicle; assumed 2, 2.5 & 3 ft for W_n of 20, 22 & 24 ft, respectively.
F_A = width of front overhang, ft.
Z = extra width allowance for difficulty of driving on curves, ft.

u = track width on tangent (out-to-out) 8.5 ft
R = radius on centerline of 2-lane pavement, ft
L = wheelbase
A = front overhang
V = design speed of highway, mph

Figure 17. Elements of Pavement Widening (1).

CALCULATED AND DESIGN VALUES FOR PAVEMENT WIDENING ON OPEN HIGHWAY CURVES
(2-LANE PAVEMENTS, ONE-WAY OR TWO-WAY)

Degree of curve	Widening, in feet, for 2-lane pavements on curves for width of pavement on tangent of: 24 feet						22 feet					20 feet			
	Design speed, mph						Design speed, mph					Design speed, mph			
	30	40	50	60	70	80	30	40	50	60	70	30	40	50	60
1	0.0	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	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	1.0	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			

NOTE: Values less than 2.0 may be disregarded.
3-lane pavements: multiply above values by 1.5
4-lane pavements: multiply above values by 2.0
Where semitrailers are significant, increase tabular values of widening by 0.5 for curves of 10 to 16 degrees, and by 1.0 for curves 17 degrees and sharper.

Table 10. Design Values for Pavement Widening (11).

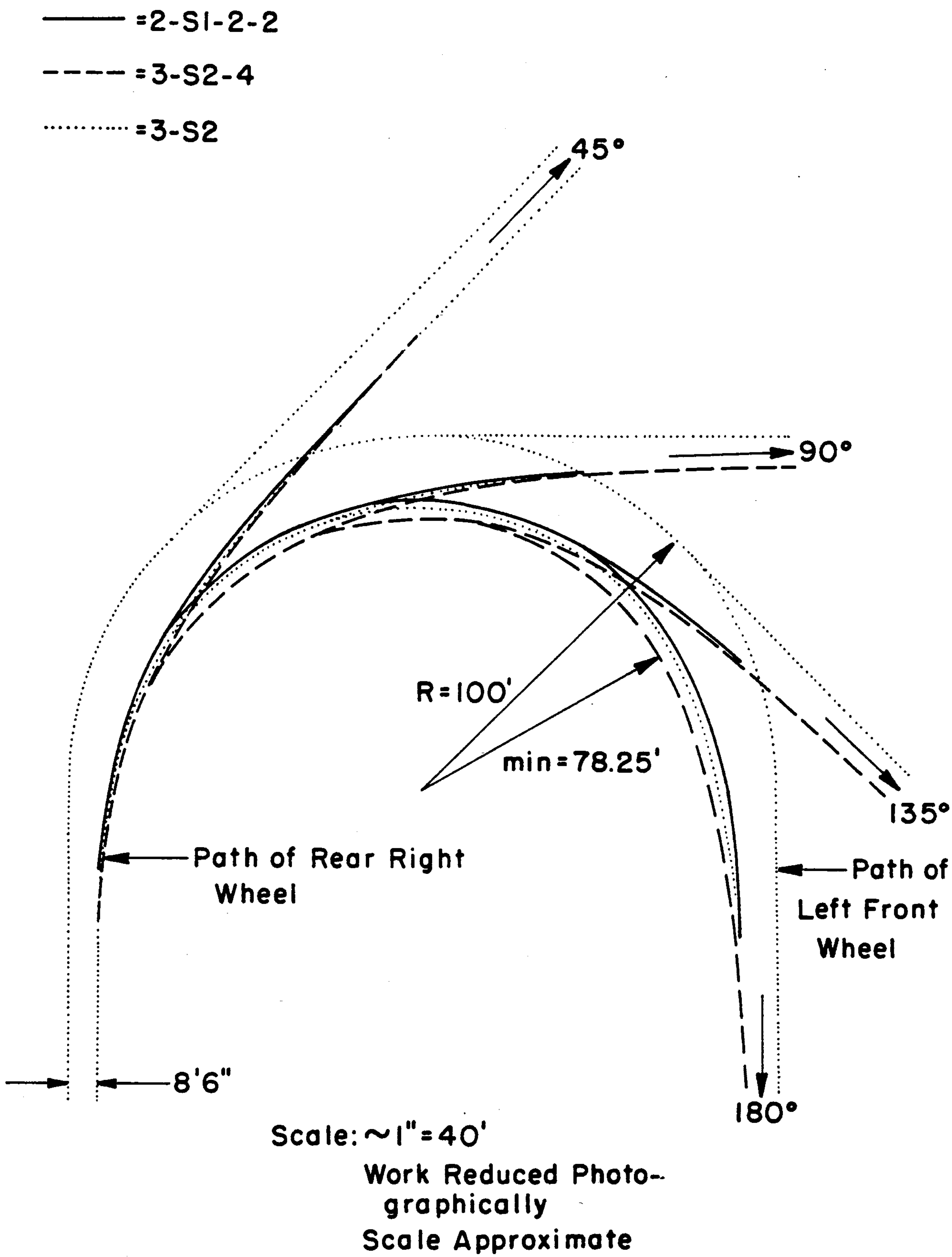


Figure 18. Off-Tracking Characteristics of Triple, Double, and Single Combination Trucks (11).

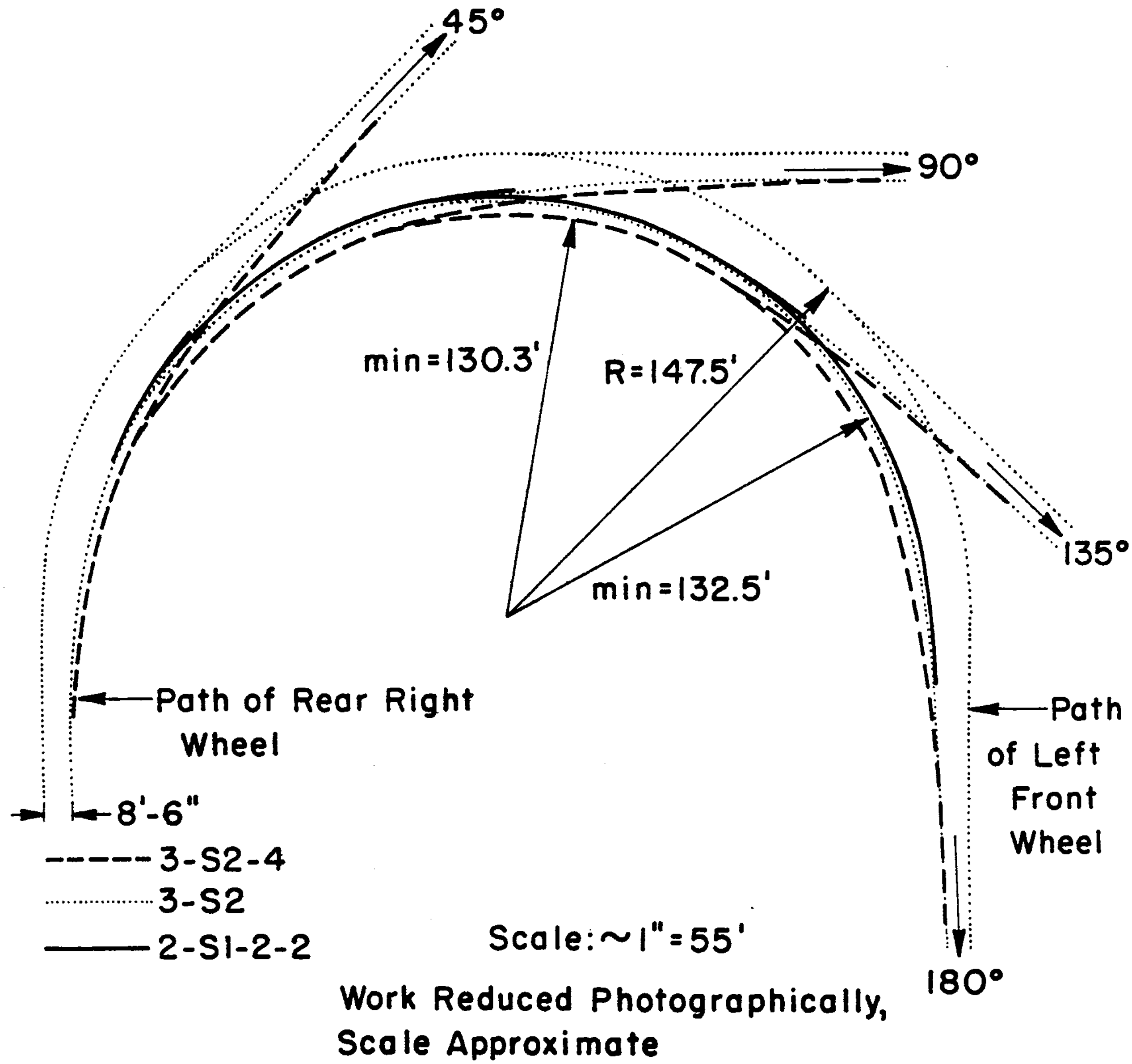


Figure 19. Off-Tracking Characteristics of Triple, Double, and Single Combination Trucks (11).

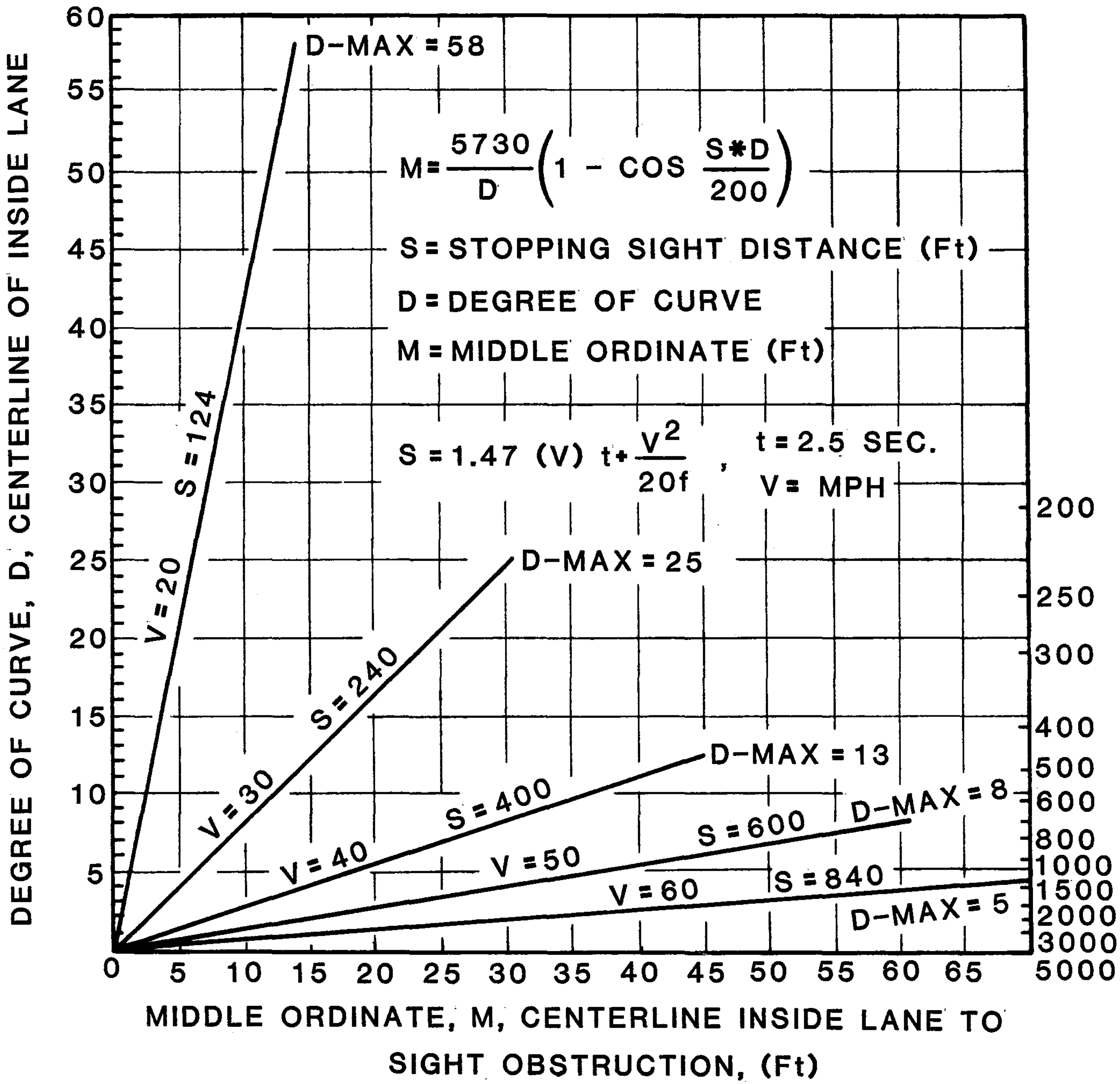


Figure 20. Required Value of the Middle Ordinate

compared to similar values of curvature and speed on the AASHTO chart shown in Figure 21. This reflects the increased stopping distance requirements of trucks as compared to passenger cars.

Larger Radii Horizontal Curves

A longer minimum radii or smaller degree of curvature can be determined by reducing the maximum friction. Suppose a designer wanted to allow for 10 percent higher level of minimum friction needed for the stability of heavy trucks. A straightforward approach to provide an additional margin of safety for these vehicles would be to reduce f_{max} by 10 percent. For example, the design speed is 70 mph and e_{max} is to be 0.08. For 70 mph, AASHTO lists f_{max} as 0.10 which would be reduced to 0.09 to provide a safety margin for trucks. Using $f = 0.09$, $e = 0.08$, and $V = 70$ mph, the minimum radius equation yields a radius of 1922 ft. If f had been 0.10, the minimum radius would have been 1815 ft. In this situation, a 10 percent decrease in f_{max} results in a 6 percent increase in the minimum radius. Additional information regarding the study findings can be found in FHWA report entitled, Side Friction for Superelevation on Horizontal Curves. (32).

VERTICAL ALIGNMENT

Vehicle Operating Characteristics on Grades

AASHTO states that trucks display up to 5 percent increase in speed on downgrades and about a 7 percent or more decrease in speed on upgrades as compared to operation on level terrain. On upgrades, the maximum speed a vehicle can maintain is dependent upon the vehicle's weight to horsepower ratio, as well as the length and steepness of the grade.

The weight to horsepower ratio of a truck is the ratio of the gross weight of the truck divided by its net horsepower. Net horsepower is the brake horsepower of the truck measured at the clutch. It does not reflect friction losses of the driveline of the truck and losses due to rolling resistance.

AASHTO uses a 300-pound per horsepower ratio as representative of the operational characteristics of trucks on grades. The previous edition of the AASHTO policy used a weight-to-horsepower ratio of 400. The reduction from 400 to 300 represents improved vehicle performance, such as maintaining higher speeds on upgrades and faster acceleration on the level terrain. The recommended change was the result of several studies which showed that the national average of weight-to-horsepower ratios declined from 360 in 1949 to 210 in 1975. It should be noted that these average values of weight-to-horsepower ratios were obtained from trucks averaging 40,000 pounds gross vehicle weight.

Other studies also indicate that the national average of weight to horsepower ratios has declined in the past few years. Walton and Gericke (11) state that the expected performance of modern day trucks will be better than the national representative trucks of the past due to superior transmissions, high torque rise engines, and the availability of bigger engines. Yu and Walton, in a study of the characteristics of double and triple combinations operating in the U.S. (3), found that most of these types of trucks were operating in the 0 to 100 weight-to-horsepower range.

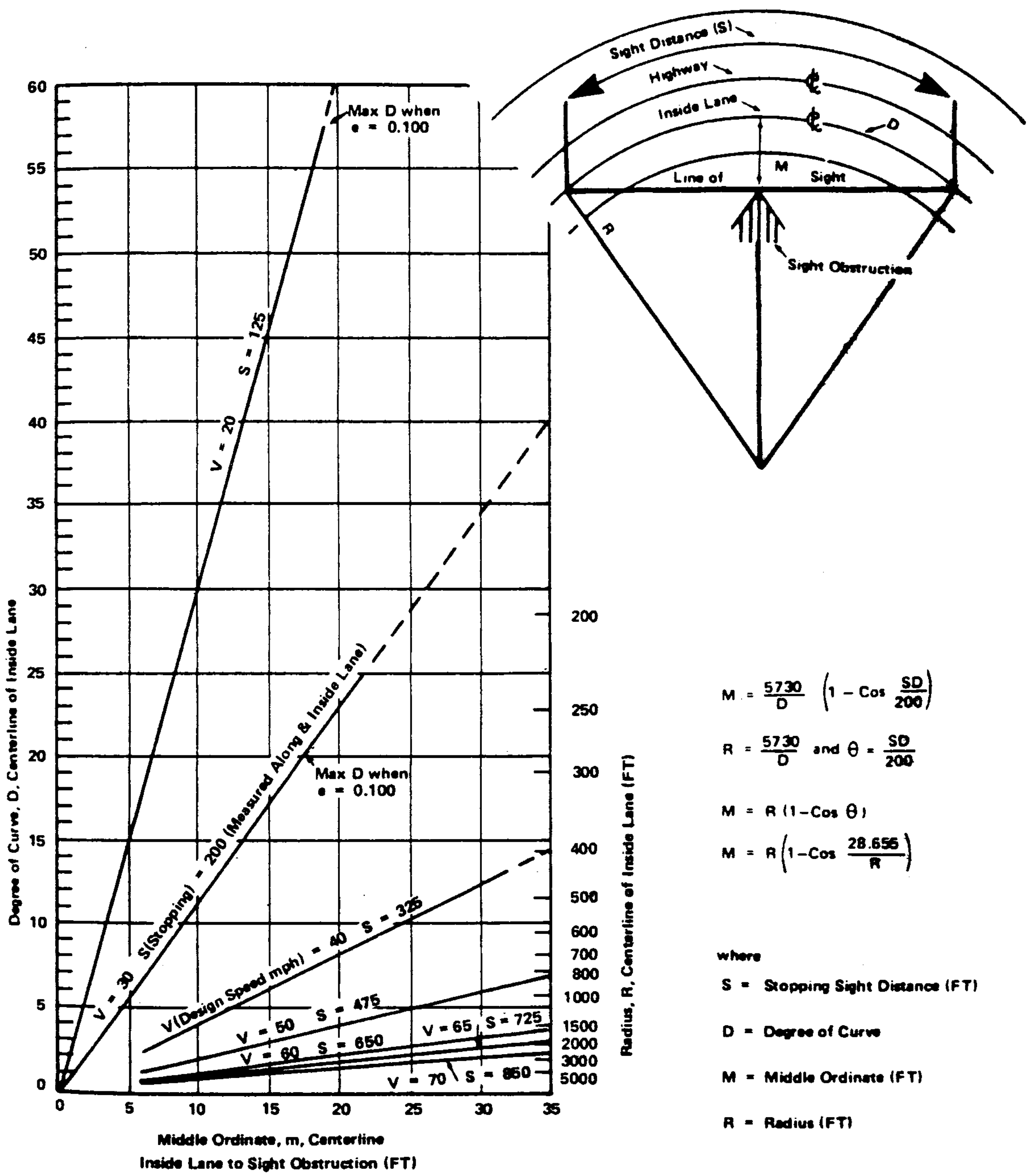


Figure 21. Middle Ordinate Value (1).

This is significantly below the 300 pound per horsepower ratio used by AASHTO.

The 300 pound per horsepower ratio recommended by AASHTO should be used for truck facility design unless studies prove that this value is not appropriate. Current speed-distance curves for 300 pound per horsepower trucks operating on various grades are shown in Figures 22 and 23.

Control Grades for Design

AASHTO Policy on Geometric Design of Highways and Streets recommends that maximum gradient be controlled by functional class, terrain, location (urban vs. rural), and design speed. The current version of AASHTO suggests maximum grades of 7 to 8 percent for a design speed of 30 miles per hour. For a 70 miles-per-hour design speed, rural conditions, and rolling terrain, the maximum gradient is 5 percent for collectors and 4 percent for arterials.

Critical Length of Grade for Design

The steepness of a grade is not the only factor which determines the ultimate crawl speed of a vehicle on a grade. The length of the grade must also be taken into account. The critical length of grade is that length of grade which will not produce an unreasonable speed reduction in the vehicle negotiating that grade.

The design vehicle used to determine the critical length of grade is assumed to be 300 pounds per horsepower as stated earlier. This is a conservative value since many trucks operate below this value and therefore exhibit improved operating characteristics as compared to the design vehicle.

In 1955 Huff and Scrivner (19) determined that the average entry speed for trucks on grades was 47 miles per hour. Walton and Lee (20) reformulated the vehicle entry speeds for 55 miles per hour in a study of truck operating characteristics on grades in the State of Texas. The new AASHTO policy has adopted this speed for entry on grades for trucks of 300 pounds per horsepower. Since 300 pounds per horsepower represents a truck with superior operating characteristics (as opposed to the 400 pound per horsepower design vehicle), it is logical to assume that the entry speeds on grades would be higher. Glennon and Joiner (21) found that a 10-mile-per-hour speed reduction as compared to other traffic on the roadway was appropriate to determine critical length of grade. AASHTO has since adopted this policy in their determination of critical grades. Critical length of grade for various speed reduction values for a 300-pound per horsepower truck is shown in Figure 24.

Climbing Lanes

AASHTO justifies the need for climbing lanes based on capacity criteria. The effect that trucks have on capacity is determined by the speed differential between the trucks and the passenger cars in the traffic stream. As the speed differential increases, the amount of delay experienced by the passenger cars increases.

For ETF design, although there are no passenger cars to jeopardize,

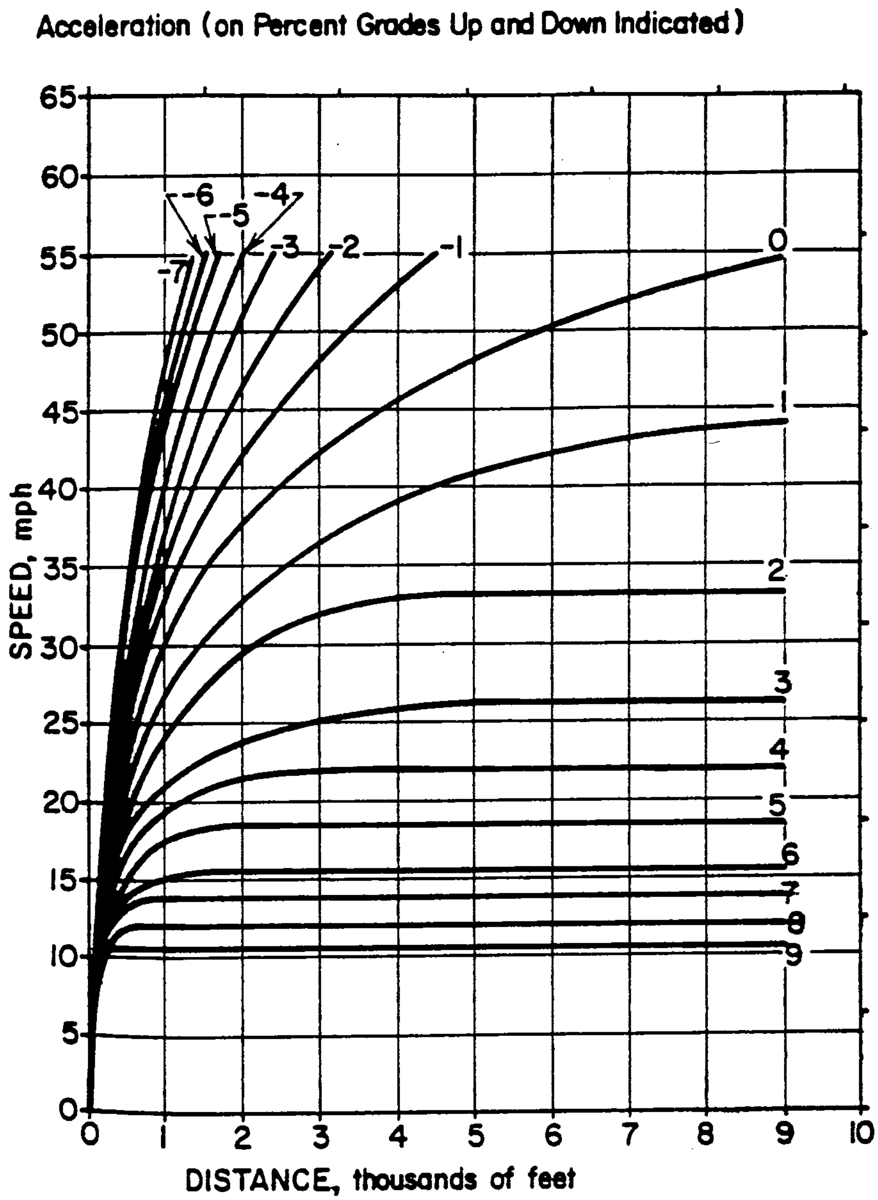


Figure 22. Speed-Distance Curves on Various Grades (1).

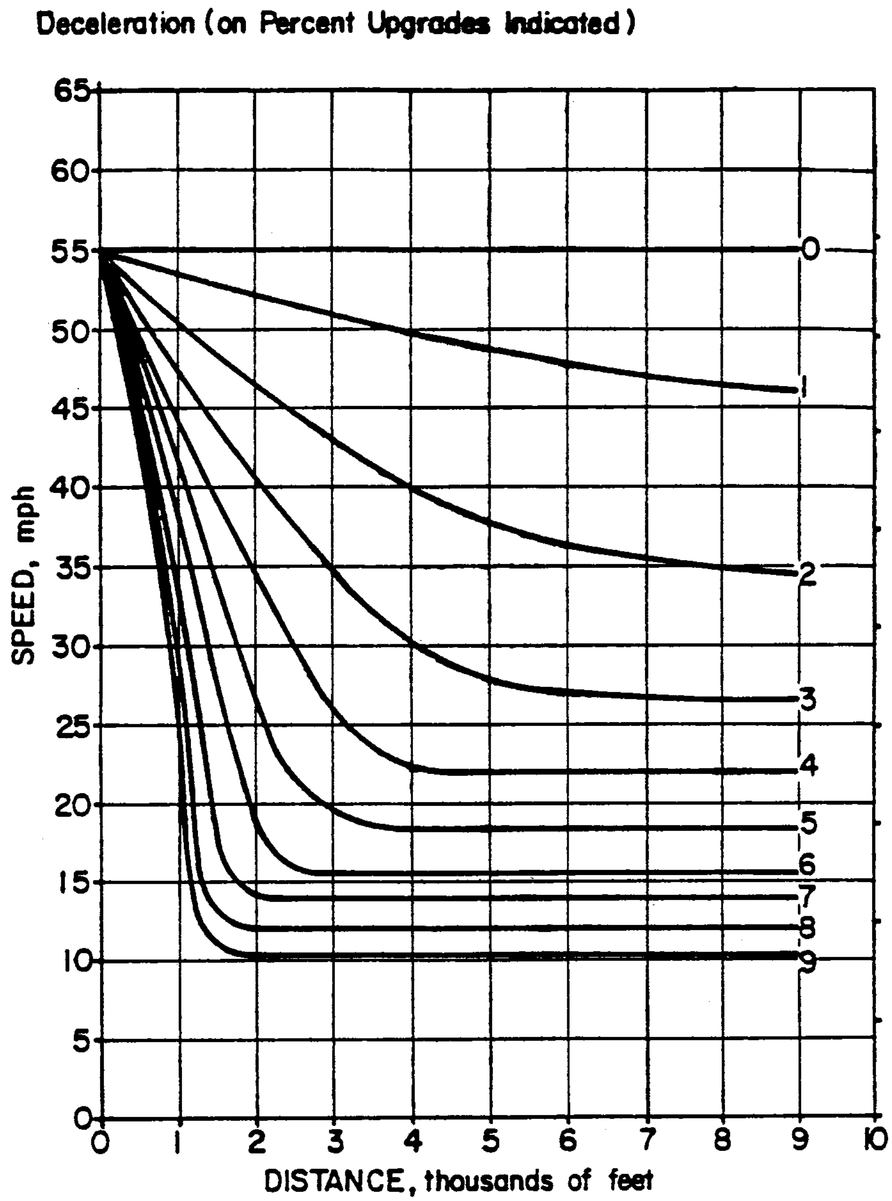


Figure 23. Speed-Distance Curves on Various Grades (1).

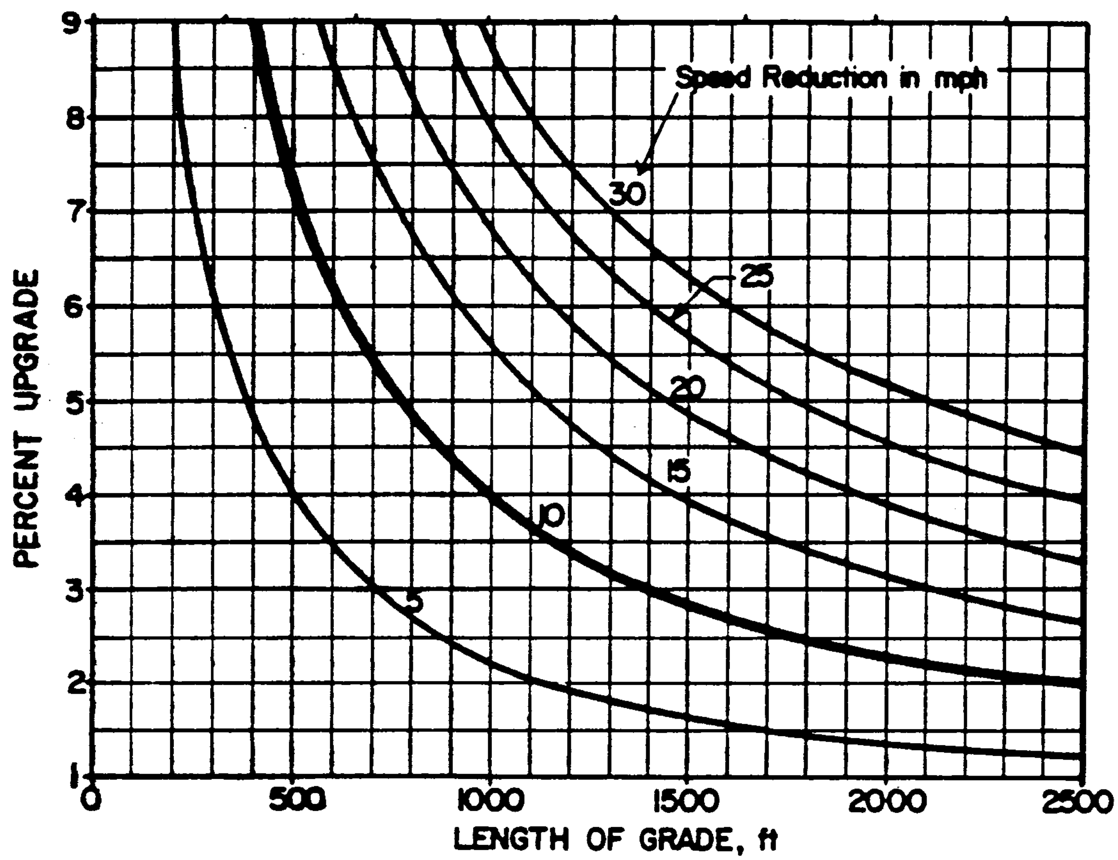


Figure 24. Critical Length of Grade for 300 lb/hp Trucks (1).

there will be empty/light cargo trucks that will be delayed by slower, heavier vehicles operating on grade. The delay experienced on grades will be a function of the difference in operating characteristics of various trucks in the traffic stream. Thus, the different types of trucks operating on these facilities should be classified according to performance characteristics so that delay due to certain slower trucks can be estimated. Some guidance may come from existing information on truck performance on grade, but how these trucks operate in a unique/isolated environment has not been established.

Emergency Escape Ramps

Occasionally, heavy vehicle operators lose control of their vehicles on long, steep descending grades as a result of brakes overheating, mechanical failure, or failure to downshift at the proper time. The construction of emergency escape ramps at these locations is desirable for the purpose of slowing or stopping these vehicles away from the main traffic stream.

Several types of emergency escape ramps are in use: the gravity-type ramp, the sand or gravel arrester bed, and gravity ramps with arrester beds. AASHTO defines four types: ascending grade, level grade, descending grade, and sandpile. The design and operation of emergency escape ramps is fairly well documented (1,22,23,24). If ETF facilities are located in areas of excessive vertical alignment, the inclusion of escape ramps should be contemplated.

Vertical Curves

Vertical curve design on highways is a function of available sight distance, comfort, drainage, and appearance. AASHTO states that minimum lengths of vertical curves determined from sight distance criteria are generally satisfactory from the standpoint of the other variables.

The basic formula for parabolic crest vertical curves used in highway design are as follows:

$$L = AS^2 / (100 ((2H_1)^{.5} + (2H_2)^{.5})^{.5} \qquad S<L \qquad (18)$$

$$L = 2S - (200 ((H_1)^{.5} + (H_2)^{.5})^{.5}) / A \qquad S>L \qquad (19)$$

Where:

L = length of vertical curve, ft;

S = sight distance, ft;

A = algebraic difference in grades, percent;

H_1 = height of eye above roadway surface, ft; and

H_2 = height of object above roadway surface, ft.

Current AASHTO values of driver eye height and height of object are 3.5 ft and .5 ft, respectively. Although truck driver eye heights were found to be approximately 8 ft for most models, this may not always be the case.

The desirable length of sag and crest vertical curves lengths for AASHTO passenger cars criteria are shown in Figure 25. Figure 26 shows the sag and crest vertical curve lengths calculated from truck stopping sight distance using a perception-reaction time of 2.5 seconds. The stopping sight distance values used in the vertical curve length calculations are shown in Table 6. A truck driver eye height of 8 ft was used for the calculation of crest vertical curves. Note, that the 8 ft driver eye height will yield shorter crest vertical lengths than for current AASHTO passenger car design criteria.

For each design speed, there is a specific truck driver eye height that will compensate for the additional stopping sight distance required by trucks. Critical truck driver eye heights at specified speeds follow:

DESIGN SPEED (mph)	FRICTION FACTOR	P-R TIME (sec)	CRITICAL TRUCK EYE HEIGHT (ft)
20	.4	2.5	5.17
25	.38	2.5	5.54
30	.35	2.5	5.93
35	.34	2.5	6.22
40	.32	2.5	6.53
45	.31	2.5	6.78
50	.30	2.5	7.01
55	.30	2.5	7.16
60	.29	2.5	7.36
65	.29	2.5	7.48
70	.28	2.5	7.65

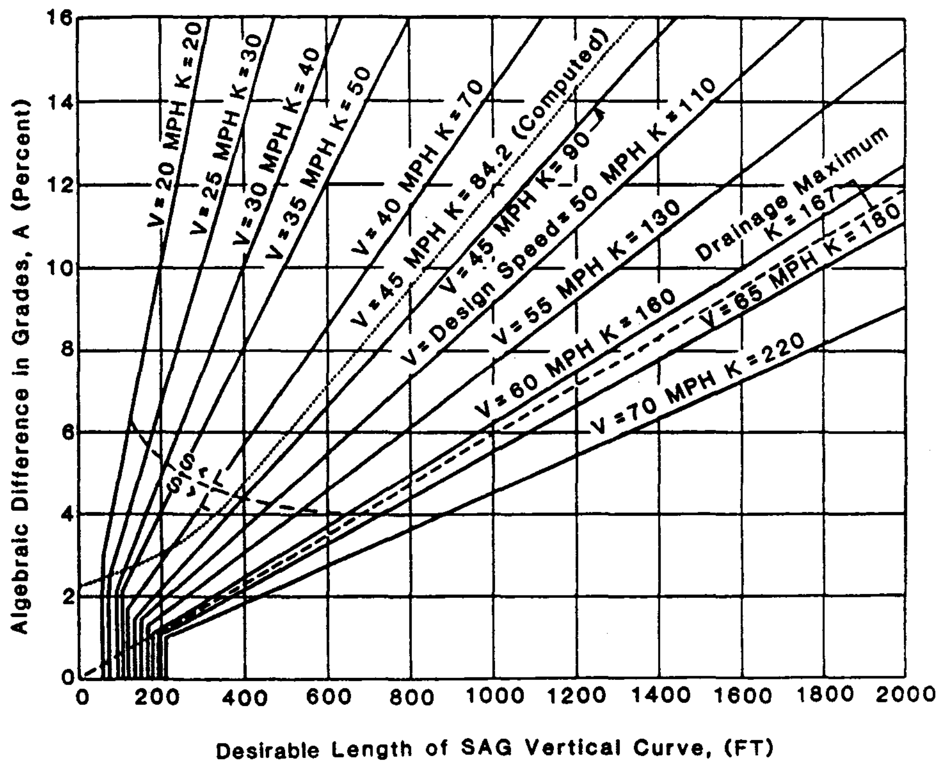
These values were obtained by equating the stopping sight distance formulas for trucks and passenger cars. Object height of 6 inches and a perception-reaction time of 2.5 seconds were used in both equations. Passenger car driver eye height was set at 3.5 ft.

After reducing the equations, the truck driver eye height value that will give a length of crest vertical curve equal to current AASHTO criteria was obtained by specifying a speed and friction value. (In other words, a truck driver eye height must be above the critical value in order to compensate for the truck's longer stopping distance.) Respective friction factors were obtained from page 138 of the Green Book (1).

The AASHTO equations for sag vertical curves are as follows:

$$L = AS^2 / (400 + 3.5S) \qquad S<L \qquad (20)$$

$$L = 2S - (400 + 3.5S) / A \qquad S>L \qquad (21)$$



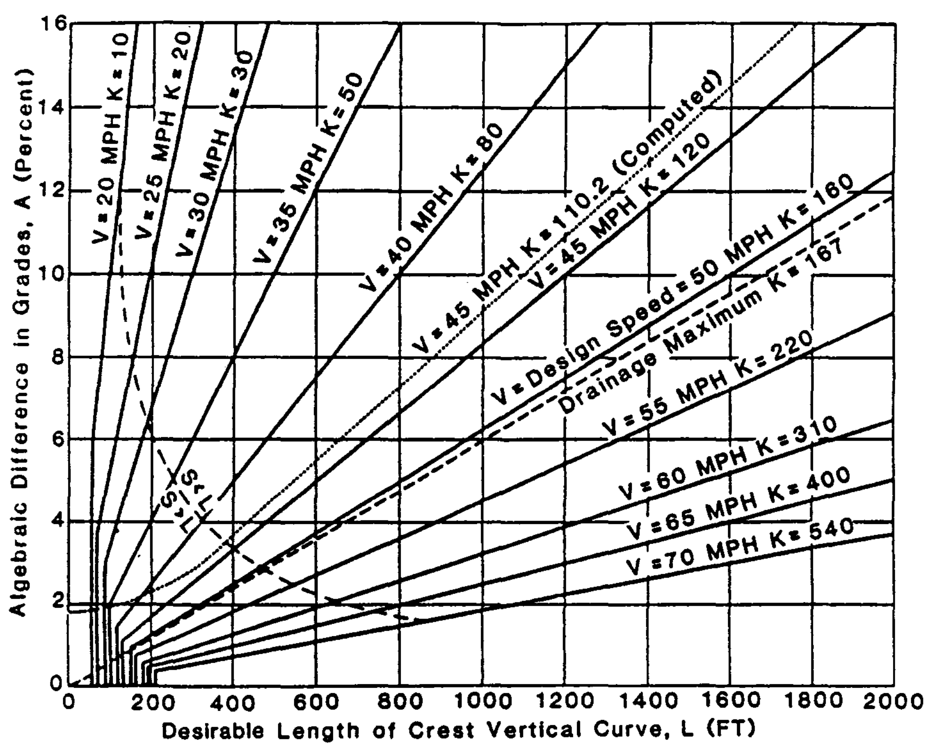
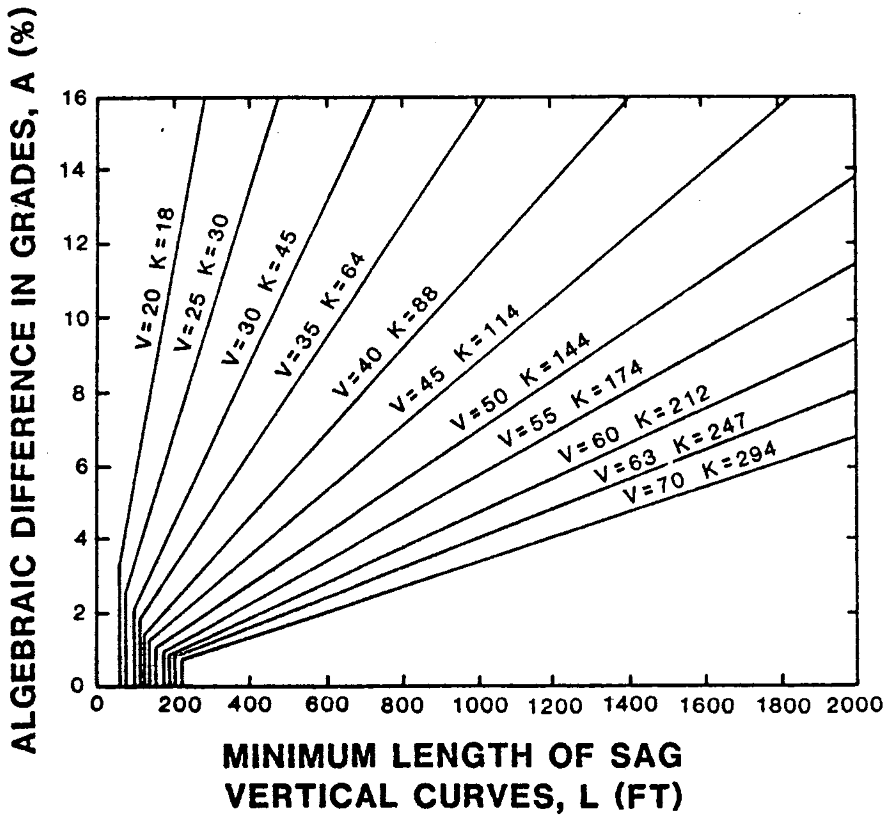


Figure 25. AASHTO Criteria for Vertical Curves (1)



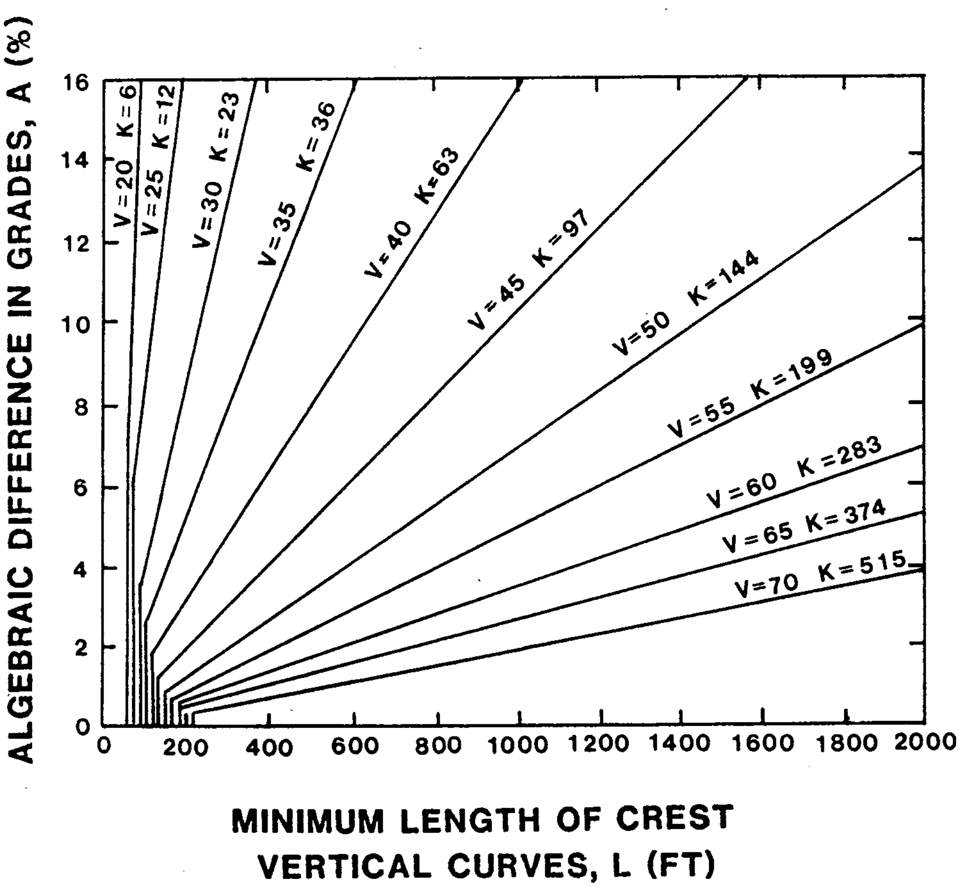


Figure 26. Vertical Curves Lengths Calculated from Stopping Sight Distance Values

Where:

L = length of sag vertical curve, ft;

S = light beam distance, ft; and

A = algebraic difference in grades, percent.

The light beam distance in the equations is a function of headlight heights. AASHTO uses a value of 2 ft for headlight heights for passenger cars. Headlight heights for trucks are generally higher than passenger cars, yielding greater light beam distances. Thus the AASHTO equations provide conservative estimates for sag vertical curve design which are suitable for use in ETF design.

CROSS SECTION ELEMENTS

Lane Widths

High-type pavements are generally required to have 10-ft to 12-ft lane widths, with 12-ft lane widths the most predominant. AASHTO considers 12-ft lane widths essential for adequate clearance of commercial vehicles on two-lane pavements. Walton and Gericke (11) have studied the possible effects of increased truck widths on the present practice of highway geometric design in the State of Texas. As a result of the Surface Transportation Assistance Act of 1982, the maximum legal vehicle width in the State was increased from 96 inches to 102 inches. Walton and Gericke recommended that pavement widths where these trucks will be operating should be widened to at least 12 ft for both two-lane and multilane highways.

A study was conducted by Canner and Hale (25) to determine vehicle encroachment on bituminous shoulders and lateral placement of vehicles within the outside driving lane of four-lane divided pavements. The vehicles studied were trucks with dual tires on the back axle, tractor-trailer combinations, and buses. The highway sections were edge striped such that the effective lane width was 12 ft. However, the pavement extended 3 ft outside the right edge stripe in some sections. At these sections, heavy vehicles moved toward or crossed over the right edge stripe more often than on sections where the edge stripe was located at the edge of the pavement.

Lee (26) conducted studies of lateral placement of trucks on four-lane divided highways with 12 ft traffic lanes. His data indicates that the largest percentage of observations of wheel placement were within 2 ft or less from the right pavement edge. As the size of the truck increased, the percentage of observations within the 2-ft distance increased. Also, the frequency of placement within the 2-ft distance increased on curved sections of roadway.

The placement of vehicles near the right edge of the traveled lanes shown in these two studies is evidence that truck drivers are not satisfied with 12-ft lane widths.

Leisch and Associates (2) have determined pavement width requirements based on an analysis of width related conflicts under critical operating situations. Using data presented by Taragin in "Effect of Roadway Width on Vehicle Operation" in 1945, they compiled a table which illustrates observed clearances on either side of a vehicle for various maneuvers on rural highways. This data is shown in Table 11. This table shows that trucks meeting other trucks on two-lane highways, with 24 ft pavement widths, prefer a 4 ft clearance between the opposing vehicle bodies and a right side body clearance of 2 ft from the edge of the pavement. For an 8-ft wide truck, this means that the truck is traveling down the center of the lane with a 2-ft clearance on both sides of the vehicle to the edge of the lane. During free moving conditions, the driver of the truck positions his vehicle 2.8 ft from the pavement edge and 1.2 ft from the centerline of the roadway. In other words, when no opposing traffic is encountered, the driver increases the clearance from the right side of the vehicle to the pavement edge. Therefore, it is apparent that trucks operating on 12-ft lane widths do not have the available width to satisfy desirable clearance on both sides of the vehicle.

Leisch states that the lateral placement of trucks meeting other trucks indicates that trucks desire at least two ft of clearance between the right side of the truck body and the edge of the pavement. With an increase in pavement width from 22 to 24 ft, only .1 ft was added to the left-side clearance. The remaining .9 ft was used to increase the average right-side clearance to about 2 ft. Although other data are not available to support this conclusion, it appears additional roadway width, if available, would be used to increase the right-side clearance. Based on this information, Leisch concluded that a right-side clearance of 2 ft represents a minimum value for trucks.

Leisch quoted Taragin in stating that the desired clearance between bodies of meeting passenger cars is about 5 ft, or 2.5 ft to the centerline. Leisch further concluded that although trucks operate with less body clearance than autos, desirable clearance between the bodies of opposing vehicles was assumed to be the same as that for autos (left-side clearance of 2.5 ft to the centerline). Thus the desirable lane widths for trucks meeting other trucks is the sum of the left- and right-side clearances and the width of the truck. For example, 102-inch wide trucks operating on a two-lane highway exclusively for trucks would require 13-ft pavement widths in each direction (8.5 ft vehicle width plus 2 ft of right clearance and 2.5 ft of left clearance). This is illustrated by Figure 27.

Taragin's report concluded that pavement widths adequate to accommodate meeting-vehicle maneuvers were more than adequate for passing maneuvers. If this is the case, then the criteria for determining the lane widths for two-lane highways is applicable to multi-lane facilities. Thus, for ETF design, the following equation could be used to determine lane widths for design purposes:

$$W = Wv + 4.5 \qquad (22)$$

	Pavement Width (Feet)			
	18	20	22	24
FREE MOVING VEHICLES (Edge of Pavement to Vehicle Body)				
Passenger Car	2.1	2.9	3.4	4.1
Truck	0.7	1.4	2.0	2.8
(Centerline to Vehicle Body)				
Passenger Car	0.8	1.0	1.5	1.8
Truck	0.3	0.6	1.0	1.2
MEETING OPPOSING VEHICLE (Between Opposing Vehicle Bodies)				
Both Passenger Cars	3.2	4.0	4.8	5.2
Passenger Car and Truck	2.6	3.5	4.5	4.8
Both Trucks	1.6	3.0	3.9	4.0
(Edge of Pavement to Vehicle Body --Meeting Truck)				
Passenger Car	1.2	1.8	2.3	3.1
Truck	0.2	0.5	1.1	2.0
PASSING ANOTHER VEHICLE (Between Vehicles)				
Both Passenger Cars	2.3	2.9	3.9	4.8
Passenger Car and Truck	2.3	2.3	2.8	3.0
Both Trucks	1.2	1.3	n.a.	n.a.
(Edge of Pavement to Vehicle Body --Outer Vehicle)				
Passenger Car	1.5	2.3	3.0	n.a.
Truck	0.5	1.1	1.5	2.2

Source: Taragin, A. "Effect of Roadway Width on Vehicle Operation," Public Roads, Vol. 24, No. 5, 1945.

Table 11. Observed Clearances Associated with Maneuvers Occurring on Rural Highways

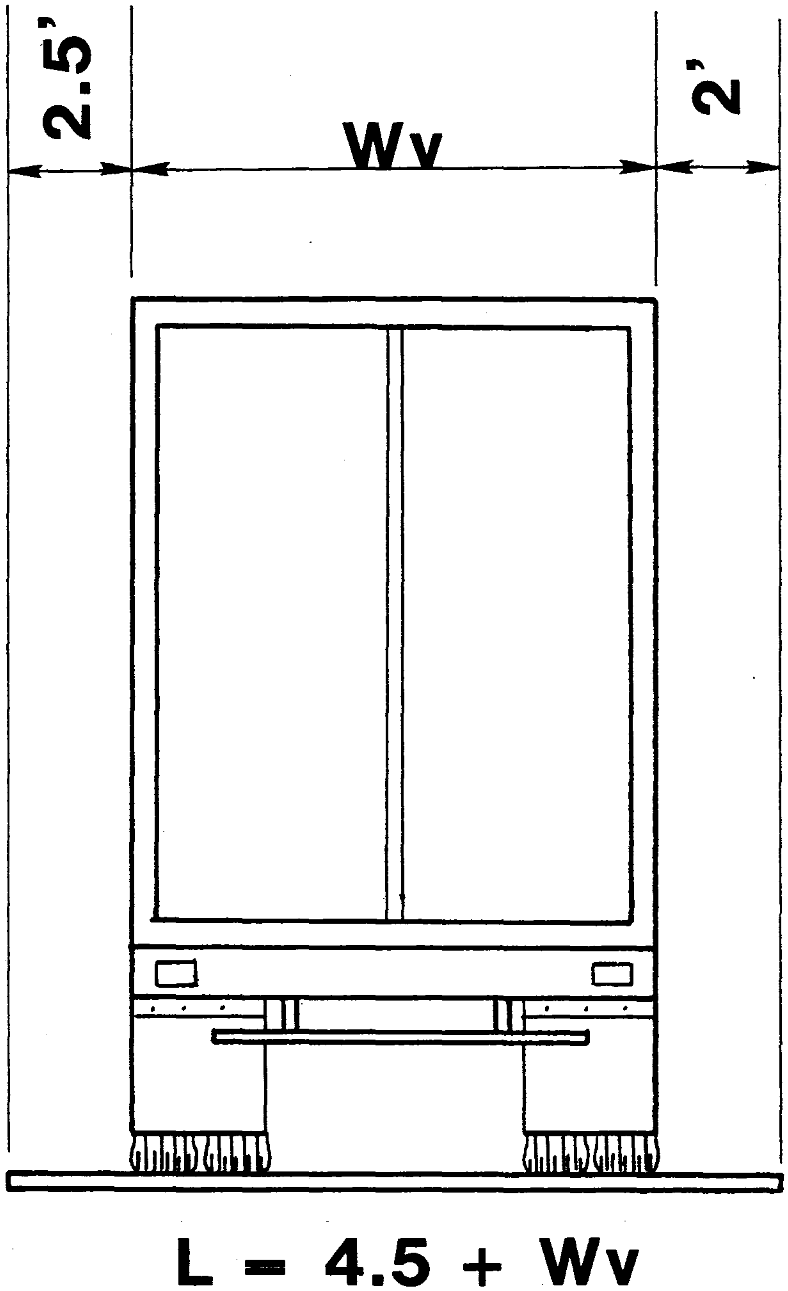


Figure 27. Clearance for Trucks

Where:

W = width of one lane, ft; and

W_v = width of the vehicle, ft.

If double and triple combinations are included in this discussion, the lateral instability of these trucks at operating speeds may seem to warrant greater pavement widths. However Peterson (10) reports that, although Utah regulations require that triple combinations not sway more than 3 inches to either side on smooth level pavement, significant swaying did occur if a strong wind was blowing, or if the trailers were loaded unevenly. A definition of "significant swaying" was not provided in the report.

Shoulder Widths

AASHTO policy provides for a desirable width of shoulder which will enable a stopped vehicle to clear the roadway by at least 1 ft and preferably by 2 ft. This has led to the adoption of usable shoulder widths of 10 ft along high-type facilities. AASHTO recommends a 12 ft shoulder along heavily traveled and high-speed facilities which carry large amounts of truck traffic.

AASHTO distinguishes between "graded" and "usable" shoulders. The graded width of a shoulder is the distance from the edge of the travelled way to the intersection of the shoulder slope and the front slope of the roadway. The usable width is that width which can be used when a driver makes an emergency or parking stop. A distance of 2 ft from the outer edge of the usable shoulder to roadside barriers, walls, or other vertical elements is recommended. Adequate shoulder widths reduce the potential for collisions with fixed obstacles, overturning of vehicles, running off the roadway, and pedestrian accidents.

Guardrails

The Bluebook states that guardrails should be used where vehicles leaving the roadway would be subject to hazard, but only if the roadside hazard constitutes a greater threat to safety than striking the guardrail itself. Guardrails are designed to redirect the impacting vehicle, reduce its velocity, and guide it along the rail as it decelerates. The current design standards for guardrails assume a design vehicle of 4500 pounds, traveling 60 miles per hour, and striking the rail at a 25 degree angle (27). No provisions for heavy vehicles are made in the design of most guardrails. As a consequence, most of the roadside hardware in existence today is proving to be inadequate for heavy vehicles such as trucks and busses (28). Facilities designed especially for heavy vehicles will require roadside hardware suitable for truck operating characteristics.

Several types of guardrails and bridgerails have successfully redirected heavy vehicles with minimal property damage. The most common is the concrete median barrier, or "safety shape." Full scale impact testing with heavy vehicles resulted in the successful restraining and redirection of the vehicle at speeds of up to 45 miles per hour and a 15-degree impact angle (29). Concrete bridge rails have also been developed for redirection of errant trucks on elevated structures (30). These rails are expensive ($41 per foot in 1980), and additional research is needed to develop less costly barriers for heavy vehicles.

Drainage Channels and Sideslopes

Drainage channels, while performing the vital task of directing water away from the highway, should not pose a serious safety hazard to errant vehicles. Extensive studies have been performed to determine optimum ditch designs for highways using passenger cars as test vehicles (31). Due to obvious cost problems, detailed studies have not been performed on the effects of ditches on the recovery of errant heavy vehicles.

Roadway sideslopes are a similar matter. Vehicle testing on sideslopes has been performed using passenger cars as test vehicles. Research is lacking on the controlability of heavy vehicles traversing roadside slopes.

Generally, roadside slopes should be as flat as possible to avoid vehicle overturning. This is also true for ditch sections. Ditches should be as wide as possible where adequate right-of-way exists. Although current standards for safe roadside cross sections were obtained using passenger car data, current criteria should be used as a "worst case" alternative. They provide a starting point in the determination of safe roadside cross sections for heavy vehicles.

SUMMARY AND CONCLUSIONS

Based on a review of existing literature, additions to current highway design policy are required in order to develop criteria for the design of exclusive truck facilities:

1. Design vehicles should be adopted to represent some of the larger and heavier trucks operating on the roadway today such as the 105-ft doubles and triples. Although these types of vehicles are not legal in the State of Texas at this time, the operation of these types of trucks may be desirable in the future.

2. The current maximum height criteria for heavy trucks is 13.5 ft. Due to industry standards, highway design practice, and in the interest of uniformity from one functional class of highway to another, no change is recommended for maximum vehicle height criteria.

3. The 102-inch vehicle width allowed for in the Surface Transportation Act of 1982 is considered appropriate for ETF design. However, if the operation of large volumes of oversize trucks is anticipated, larger representative vehicle widths should be used. Determination of optimum design vehicle width is largely a question of economics.

4. The minimum length of design vehicle to be used for design purposes is 65 ft, as this reflects current maximum vehicle lengths allowed in Texas. As stated earlier, it may become more desirable to allow longer trucks to operate on the system in the future; therefore consideration should be given to adopting a design vehicle length in excess of current legal standards.

5. Driver eye height for heavy vehicles was found to range from an average of 7.84 ft for cab-behind-engine configurations to 8.41 ft for cab-over-engine configurations. The current AASHTO value for driver eye height is 3.5 ft, which is not representative of trucks in general. This leads to excessively long vertical curves when truck stopping sight distances are used.

6. AASHTO uses a value of 300 pounds per horsepower in the determination of operating characteristics of heavy trucks. This represents improved performance of heavy trucks due to increases in horsepower. It is a reasonable estimate of horsepower characteristics of heavy vehicles at this time and should be used for the design of exclusive truck facilities until future research indicates a more appropriate value.

7. Truck braking tests have shown that cars stop in two-thirds the distance required by heavy trucks. Based on these findings, the AASHTO braking distance equation has been modified to represent braking characteristics of heavy combinations. Truck braking distances are greater than those predicted by AASHTO for passenger cars and are recommended for use in ETF design.

8. The 2.5 second reaction time used by AASHTO may need to be changed for ETF design to 3.2 seconds. This value represents the recent findings in driver behavior research which is the 85th percentile value for the driving population.

9. Stopping sight distance values have been calculated for truck facility design to account for the increased stopping distance requirements and the 3.2 driver perception-reaction time. These values are larger than the current values provided by AASHTO.

10. Passing sight distance requirements for heavy trucks passing other heavy trucks have been determined for various speeds. The values derived assume that both vehicles are 65 ft in length. The equations used can be modified to reflect various speed or length combinations.

11. For horizontal curves, AASHTO design criteria for high speed facilities appears adequate for ETF design. This assumes that the rollover thresholds of heavy vehicles are in the .25 to .4 g range. Pavement widening on curves, as determined by AASHTO, is adequate for present vehicle lengths. However, if longer design vehicles are used (> 65 ft), the AASHTO values will need to be recalculated. The values of the middle ordinate for horizontal curve sight distance design have been recalculated based on truck stopping distance values.

12. In areas where long, steep grades result in runaway truck incidents, truck escape ramps should be provided. Four types of ramps are in use: the gravity or ascending grade, the level grade, descending grade, and the sandpile. Most are easily placed in the right-of-way areas of the facilities.

13. An 8 ft truck driver eye height yields shorter crest vertical lengths than for current AASHTO passenger car design criteria. However, for each design speed, there is a specific truck driver eye height that will compensate for the additional stopping sight distance required by trucks.

14. Lane widths for exclusive truck facilities should be at least 12 ft in width, preferably 13 ft. The wider lane widths may be used if sufficient funds permit the increased costs associated with increased pavement widths. Shoulder widths should be about 10 to 12 ft where possible to allow 1 to 2 ft of clearance between a stopped vehicle and the pavement edge.

15. Concrete median barriers should be used to contain errant vehicles within the roadway. Many existing barriers are not of sufficient strength to redirect an errant heavy vehicle. Research into bridge and roadside barriers has produced several types of barriers which will perform adequately when struck by large trucks.

16. Vehicle testing on sideslopes has been limited mostly to passenger cars. Until more is known on this subject, roadway sideslopes should consider current AASHTO guidelines.

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