Appendices to NCHRP Rpt. 400

Pt 1 of 2

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT ???

DETERMINATION OF STOPPING SIGHT DISTANCES

DANIEL B. FAMBRO, KAY FITZPATRICK, RODGER KOPPA
Texas Transportation Institute
Texas A&M University
College Station, Texas

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AREAS OF INTEREST

Geometric Design, Driver Behavior,
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(Highway Transportation)

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APPENDIX A -

BACKGROUND

One of the most important requirements in highway design is the provision of adequate stopping sight distance at every point along the roadway. Horizontal and vertical curves can limit available sight distance; however, when designed according to AASHTO (American Association of State Highway and Transportation Officials) criteria, adequate stopping sight distance will be available at every point along the curve; therefore, the design of vertical and horizontal curves is dependent upon providing the required stopping sight distance.

This appendix discusses the development of the AASHTO design equations and parameters used in calculating stopping sight distance and length of horizontal and vertical curves. It also presents the results of a sensitivity analysis of the AASHTO design parameters and a functional analysis of the resultant stopping sight distances. The appendix concludes with a discussion of alternative and international stopping sight distance models.

AASHTO STOPPING SIGHT DISTANCE MODEL

Stopping sight distances are calculated using basic principles of physics and the relationships between various design parameters. AASHTO defines stopping sight distance as the sum of two components: brake reaction distance (distance traveled from the instant the driver detects an object to the instant he applies the brakes), and the braking distance (distance traveled from the instant the driver applies the brakes to when he decelerates the vehicle to a stop) (*1*). Minimum and desirable stopping sight distances are calculated with the following equation:

SSD = Brake Reaction Distance + Braking Distance

or more specifically,

$$SSD = 0.278Vt + \frac{V^2}{254f} \qquad [1]$$

where: SSD = stopping sight distance, m;
V = design or initial speed, km/h;
t = driver perception-reaction times; and
f = friction between the tires and the pavement surface.

Roadway grade also affects stopping sight distance, i.e., stopping distances decrease on upgrades and increase on downgrades. Grade effects are calculated with the following equation:

$$SSD = 0.278Vt + \frac{V^2}{254\ (f \pm g)} \qquad [2]$$

where: g = percent grade/100, + for upgrades and - for downgrades.

Stopping sight distance on vertical curves is based on the average grade (g) over the braking or deceleration distance.

The minimum length of vertical curves is controlled by the required stopping sight distance, driver eye height, and object height. This required length of curve is such that at a minimum, the stopping sight distance calculated by Equation (2) is available at all points along the curve. Refer to the following formulas for determining the required length of crest and sag vertical curves.

For crest vertical curves:

$$L = \frac{AS^2}{200(\sqrt{h_1} + \sqrt{h_2})^2} \qquad when\ S < L \qquad [3]$$

and

$$L = 2S - \frac{200(\sqrt{h_1} + \sqrt{h_2})^2}{A} \qquad when\ S > L \qquad [4]$$

where: L = required length of vertical curve (m);
S = sight distance (m);
A = algebraic difference in grade, percent;
h_1 = eye height above the roadway surface (m); and
h_2 = object height above the roadway surface that is hidden from the driver's view (m).

For sag vertical curves:

$$L = \frac{AS^2}{2(h_3 + S\tan\theta)} \qquad when\ S < L \qquad [5]$$

and

$$L = 2S - \frac{2(h_3 + S\tan\theta)}{A} \qquad when\ S > L \qquad [6]$$

where: h_1 = height of vehicle headlights above the roadway surface (m); and
θ = upper divergence angle of headlight beam (most countries use 1°; some countries use 0°).

The curvature of a crest and sag vertical curve is often characterized by the K-factor, defined as the length of the vertical curve to effect a 1.0 m difference in grade, i.e., the length of vertical curve divided by its algebraic difference in grade. The following equation expresses K:

$$K = \frac{L}{A} \qquad [7]$$

Historical Development

Even though the basic stopping sight distance model has remained the same for the past 50 years, AASHO and AASHTO publications have addressed several changes in design parameter values within the model during that time. Engineering textbooks addressed the fundamental principles of highway design as early as 1921; however, it was not until 1940 that AASHO published seven highway design documents and formally recognized policies on certain aspects of geometric design. In that same year, these seven documents were reprinted and bound as a single volume entitled *Policies on Geometric Design* (*2*).

These policies were revised and amended in a 1954 document, *A Policy on Design of Rural Highways* (*3*). This document was revised and republished under the same title in 1965 and 1972; however, it was called the "Blue Book" because of the color of the cover (*4*). The 1994 AASHTO policy and its 1984 (*5*) and 1990 (*6*) predecessors were entitled *A Policy on Geometric Design of Highways and Streets*, and are commonly called the "Green Book." The 1994 document is the first AASHTO design policy in metric units. The changes in parameter values in the stopping sight distance model and minimum curve length equations that have occurred from 1940 to the present are summarized in Table A-1 and discussed in subsequent sections.

Design Speed. The use of design speed in calculating stopping sight distance was first adopted by AASHO in 1940. Design speed was defined as the maximum uniform speed adopted by the faster group of drivers, but not necessarily the small percentage of reckless drivers. In 1954, AASHO approximated the assumed speed on wet pavements as a percentage (85 to 95 percent) of the design speed. This reduction was based on the assumption that most drivers will not travel at full design speed when pavements are wet. In 1965, AASHO changed the approximated speed on wet pavements to be a percentage varying from 80 to 93 percent of the design speed. Several researchers have questioned the premise that drivers travel at lower speeds on wet pavements. For example, Knasnabis and Tadi suggested using design speed or an intermediate speed (average of design speed and assumed speed) to compute required stopping sight distances (*7*).

AASHO published *A Policy on Design Standards for Stopping Sight Distance* (*8*) in 1971. This policy introduced a range of design speeds defined by a minimum and a desirable value used for computing stopping sight distance. The minimum value was based on a percentage varying from 80 to 93 percent of the design speed (1965 assumed speeds on wet pavements), and the desirable values were based on the design speed. AASHTO retained the minimum and desirable values in their 1994, 1990, and 1984 policies, but noted that "recent observations show that many operators drive just as fast on wet pavements as they do on dry pavements" (*1*).

Perception-Reaction Time. Perception-reaction time is the summation of perception and brake reaction time. Brake reaction time was assumed as 1 second in 1940 (*2*); since then, the recommended value for brake reaction time has not changed. In 1940, total perception-reaction time ranged from 2 to 3 seconds depending upon design speed. In 1954, the "Blue Book" (*3*) adopted a policy for a total perception-reaction time of 2.5 seconds for all design speeds. The "Blue Book" stated "available references do not justify distinction over the range in design speed." No "available references" were cited; therefore, the reason for this change is not clear.

Design Pavement/Stop Conditions. The basic assumption in calculating braking distances since the 1940s has been a passenger car on a wet pavement with locked-wheel tires throughout the braking maneuver. Wet rather than dry pavement conditions are assumed for design because they result in lower coefficient of frictions and longer braking distances. Several researchers have questioned the locked-wheel braking assumption in the literature.

Olson et al. (*9*) stated that "locked-wheel stopping is not desirable and it should not be portrayed as an appropriate course of action." Their research assumed a controlled stop in which the driver "modulates his braking without losing directional stability and control" and used numerical integration to calculate recommended braking distances. Implicit in such a recommendation is the assumption that drivers' can control their vehicle's braking in a stopping situation and avoid locked-wheel braking.

Friction values should be characteristic of variations in vehicle performance, pavement surface condition, and tire condition. As shown in Table A-1, the friction factors were revised according to the prevailing knowledge of the time. Because of the lack of extensive field data, the 1940 AASHO (*2*) used a 1.25 factor of safety to encompass the variability in assumed friction values. The use of empirical friction factors increased as more studies were completed. Note that friction factors always decreased with an increase in speed. This phenomenon became known as a speed gradient.

TABLE A-1. History of AASHTO Stopping Sight Distance Parameters.

Parameters	1940 A Policy on Sight Distance for Highways	1954 A Policy on Geometric Design of Rural Highways	1965 A Policy on Geometric Design of Rural Highways	1971 A Policy on Design Standards for Stopping Sight Distances	1984 and 1990 A Policy on Geometric Design of Highways and Streets
Assumed Speed	Design Speed	85 to 95 percent of design speed	80 to 93 percent of design speed	Minimum - 80 to 93 percent of design speed Desired - design speed	Minimum - 80 to 93 percent of design speed Desired - design speed
Perception - Reaction Time	Variable: 3.0 sec at 30 mph 2.0 sec at 70 mph	2.5 sec	2.5 sec	2.5 sec	2.5 sec
Design Pavement/ Stop Condition	Dry Pavement Locked-wheel	Wet Pavement Locked-wheel	Wet Pavement Locked-wheel	Wet Pavement Locked-wheel	Wet Pavement Locked-wheel
Friction Factors	Ranges from 0.50 at 30 mph to 0.40 at 70 mph	Ranges from 0.36 at 30 mph to 0.29 at 70 mph	Ranges from 0.36 at 30 mph to 0.27 at 70 mph	Ranges from 0.35 at 30 mph to 0.27 at 70 mph	Slightly higher at higher speeds than 1970 values
Eye Height	4.5 ft	4.5 ft	3.75 ft	3.75 ft	3.5 ft
Object Height	4.0 in	4.0 in	6.0 in	6.0 in	6.0 in

Driver Eye Height. Driver eye height values are a combination of driver stature and driver seat height. The design value for driver eye height is selected so that most driver eye heights in current vehicles will be greater than the design value. As shown in Table A-1, this design parameter has decreased from 4.5 to 3.5 feet over the past 50 years. The change in eye height can be attributed to the increase in number of small vehicles, changes in vehicle design, and changes in driver seat design. The design eye height was based on the prevailing distribution of drivers and vehicles at the time of each AASHTO publication. The most significant decrease in driver eye height took place between 1954 and 1965, when the eye height changed from 4.5 to 3.75 feet. Although the trend has been a continuing decrease in eye height, most studies now state that the eye height will not decrease significantly in the future (*9, 10*).

It should be noted that a truck driver's eye height is much higher than a passenger car driver's eye height because of the differences in seat heights. At crest vertical curves this higher eye height partially compensates for longer truck braking distances; however, the benefits of higher eye heights are lost on horizontal curves unless the truck driver can see over lateral obstructions.

Object Height. Over the past 70 years the issue of which object height to use in calculating stopping sight distance has been a much-discussed subject. Table A-1 illustrates the changes in the design object height from 1940 to the present. The object height was set the same as the driver eye height, 5.5 feet in a 1921 highway engineering textbook (*11*); however, in 1940, AASHO adopted a four-inch object height as an "average" control value (*2*). They stated that "the stationary object may be a vehicle or some high object, but it may be a very low object such as merchandise dropped from a truck or small rocks from side cuts" (*2*). The surface of the roadway would have provided the safest design, but an object height of 4 inches was chosen because large holes in modern pavements were not common and other smaller objects would be easy to avoid.

In 1954, the four-inch object height was justified as "the approximate point of diminishing returns" (*3*). The use of a zero object height was not justified because of the undue construction costs, and an object height higher than 4 inches would exclude lower hazards and produce dangerously short lengths of vertical curves. AASHO noticed that the connection between object height and vertical curve length displayed a significant relationship: the length of the vertical curve decreased rapidly as the object height increased from 0 to 4 inches. Specifically, required lengths of curves decreased by 40 percent when the object height changed from 0 to 4 inches, but by only 50 to 60 percent when the object height changed from zero to a height of more than 4 inches.

AASHO adopted a six-inch object height in 1965 (*4*); however, the use of the six-inch object is not well supported in the literature. In fact, the exact paragraph used to justify a four-inch object height in 1954 was also used to justify the six-inch object height in 1965 (*3, 4*). The 1984 and 1990 "Green Books" (*5, 6*) considered a six-inch object height to be "representative of the lowest object that can create a hazardous condition and be perceived as a hazard by a driver in time to stop before reaching it." They also add that the six-inch object is an arbitrary rationalization of possible hazardous objects and a driver's ability to perceive and react to a hazardous situation.

Olson et al. (*9*) recommended reducing the object height to 4 inches, reasoning that increasing the number of small vehicles, is causing the average vehicle clearance level to decrease. Olson's rationale was that a four-inch object is less likely to damage or deflect a vehicle than the current six-inch object; therefore, a vehicle is more likely to safely pass over a four-inch object.

Headlight Height and Angle of Divergence. When using headlight sight distance to establish sag vertical curve lengths, a headlight height of 2.0 feet and a 1.0 degree upward divergence of the light beam are generally used for design (*5, 6*). Headlight heights are first mentioned as measuring about 2.0 feet above the pavement surface in the 1940 policy; however, sag vertical curves were not mentioned at that time (*2*). In 1954, headlight sight distance appears as one of the design criterion for establishing sag vertical curve length. Length requirements were based on a headlight height of 2.5 feet and a 1.0 degree upward divergence of the light beam (*3*). By 1965, the design headlight height had been reduced to 2.0 feet, and has remained at that value since that time (*4*). No documented reason was found for the change from 2.5 to 2.0 feet, but it is consistent with the decreasing of the driver eye height because of decreasing vehicle size during this period.

Middle Ordinate. When designing a horizontal curve, the sight line is a chord of the curve, and the applicable stopping sight distance is measured along the centerline of the inside lane around the curve (*1*). The required middle ordinate value—distance from the centerline of the inside lane to the sight distance obstruction—is the criterion most important in providing acceptable stopping sight distance. Calculation of the required middle ordinates for clear sight areas at various degrees of curve is an application of simple geometry that is first mentioned in the 1940 AASHO policy (*2*). Although the basic methodology has not changed, the required stopping sight distance has increased due to changes in parameter values with the SSD model. The result of this increase is larger middle ordinate values.

Sensitivity Analysis

As discussed in the previous sections, the AASHTO design equations used six parameters to determine required stopping sight distances at vertical and horizontal curves—initial vehicle speed, perception-reaction time, coefficient of friction, driver eye height, object height, and headlight height and angle of divergence. Knowing the effect of changing one parameter's value on required stopping sight distances and vertical and horizontal curve lengths is important. Sensitivity of curve lengths to changes in each of the six design parameters is discussed in the following sections. The sensitivity analyses were conducted by holding all parameters except the one under study, at the value recommended by the 1990 AASHTO policy (*6*). Table A-2 illustrates the AASHTO recommended value for each parameter in the analysis.

Vehicle Speed. Vehicle travel speed is an extremely sensitive parameter in the determination of required stopping sight distances. Farber (*10*) noted that small deviations in speed are equivalent to large deviations in stopping sight distance. For example, at an initial speed of 60 mph, each 1-mph change in speed results in a 17-foot change in stopping sight distance. This increase is significant in vertival and horizontal curve design. Woods (*12*) noted that between 40 and 65 mph, a 10 percent increase in vehicle speed resulted in a 40 percent increase in crest vertical curve length.

Use of a design or Khasnabis and Tadi's intermediate speed (*7*) instead of AASHTO's minimum design speed, would result in longer stopping sight distances. The longer stopping sight distances would result in longer vertical and horizontal curves. Khasnabis and Tadi analyzed the sensitivity of various design parameters by finding the change in rate of vertical curvature (K-value) as opposed to finding the change in vertical curve length. An increase in K results in longer curve lengths and additional stopping sight distance. They noted that at a design speed of 70 mph, a 6 mph increase in speed resulted in a 62 percent increase in K.

Perception-Reaction Time. As mentioned previously, perception-reaction time is 2.5 seconds for all design speeds (*1*). Woods (*12*) noted that any change in perception-reaction time is actually a change in the distance traveled at the design speed. Glennon (*13*) noted that for "higher speeds, the stopping sight distance is significantly increased for a one-second increase" in perception-reaction time. Farber (*10*) also noted that at higher speeds "a small increase in reaction time has a substantial effect on stopping sight distance."

TABLE A-2. AASHTO Design Values Used in the Sensitivity Analysis.

Variable	Parameter Value	
Vehicle Speed and Coefficient of Friction	70 mph	0.26
	60 mph	0.28
	50 mph	0.30
Perception-Reaction Time	2.5 sec	
Driver Eye Height	3.5 ft	
Object Height	0.5 ft	
Headlight Height	2.0 ft	
Angle of Divergence	1°	
Algebraic Difference in Grade	6 %	

Figure A-1 illustrates required stopping sight distance and lengths of vertical and horizontal curves based on various driver perception-reaction times, vehicle speeds of 50, 60, and 70 mph and an algebraic difference in grade of 6 percent. Note that an increase in perception-reaction time always results in an increase in curve length; and at higher speeds, an increase in perception-reaction times has a greater impact at high speeds than at does at low speeds. Although not shown in Figure A-1, this effect is most obvious with larger algebraic differences in grade. The differences in curve length among the three design speeds also increase as perception-reaction time and algebraic difference in grade increase.

Hooper and McGee (*14*) noted that stopping sight distance is less sensitive to changes in perception-reaction time at higher speeds. Their reasoning being "the braking distance component accounts for a greater portion of the total distance as speed increases." In other words, vehicles travel farther during braking than during perception-reaction at high speeds. Table A-3 illustrates stopping distances and braking distances for design speeds between 30 and 70 mph. Note that 56 percent of the total stopping distance is the distance traveled during perception-reaction time at 30 mph and that this percentage decreases as vehicle speed increases; i.e., 31 percent of the total stopping distance is the distance traveled during perception-reaction time at 70 mph.

Coefficient of Friction. Regarding its effect on required stopping sight distances tire-pavement friction is the most sensitive of the six parameters. Farber (*10*) noted that "as design speed increases so does the sensitivity of stopping sight distance to pavement friction." At 50 mph, the stopping sight distance increases by 9 feet with a 0.01 decrease in friction coefficient. Woods (*15*) noted that the tire-pavement friction variable is "by far the most critical value in the determination of vertical curve length." For friction values near 0.35, he reported a 4 percent increase in vertical curve length for each 0.01 percent decrease in pavement friction.

Note that curve lengths increase at a greater rate at lower friction values. Thus, the greatest sensitivity is at the lower friction values. For friction values near 0.10, a change of 0.01 in the friction factor results in a 20 percent change in vertical curve length. Figure A-2 illustrates the stopping sight distance and resultant curve length for a range of friction values, vehicle speeds of 50, 60 and 70 mph and an algebraic difference in grade of 6 percent. Note that similar to the perception-reaction time analysis, differences in curve length between speeds increase as algebraic difference in grade increases.

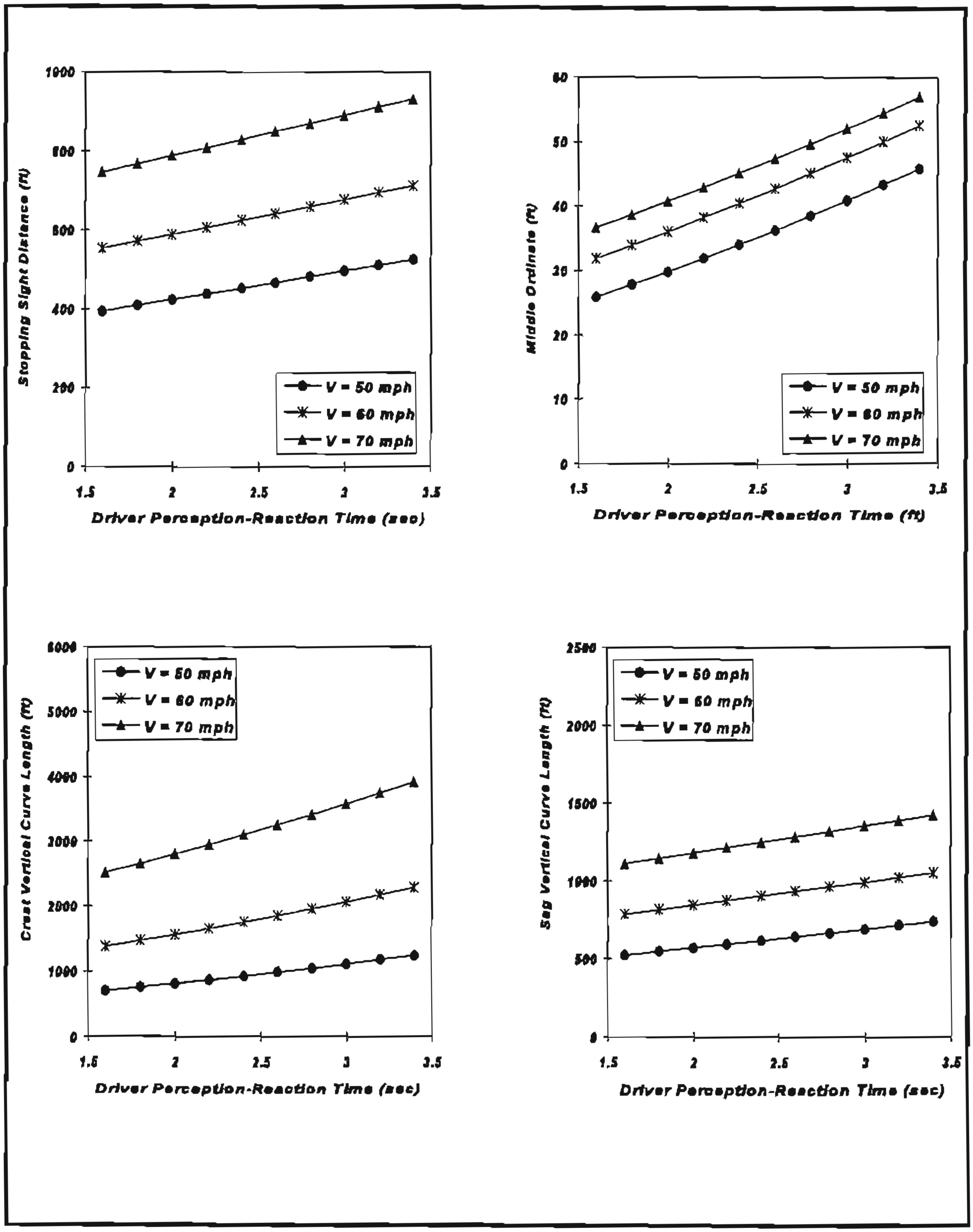


FIGURE A-1. Sensitivity of Stopping Sight Distance and Curve Geometry to Changes in Driver Perception-Reaction Time.

TABLE A-3. Comparison of Perception-Reaction and Braking Distances.

Speed (mph)	Stopping Sight Distance (ft)	Dist. Traveled During P-R (ft)	Percentage of Stopping Sight Distance	Dist. Traveled During Braking (ft)	Percentage of Stopping Sight Distance
30	196	110.3	56	85.7	44
40	314	147.0	47	166.7	53
50	462	183.8	40	277.8	60
60	634	220.5	35	413.8	65
70	841	257.3	31	583.3	69

Source: Reference (14).

Driver Eye Height. Many studies have examined the sensitivity of curve geometry to changes in driver eye height. AASHTO (*1*) noted that the change in eye height from 3.75 feet to 3.5 feet has the effect of "lengthening minimum crest vertical curves by approximately 5 percent, thereby providing 2.5 percent more sight distance." Farber (*10*) determined that "a six-inch change in driver eye height will produce a 5 percent change in sight distance." Khasnabis and Tadi (*7*) concluded that a three-inch reduction in eye height (3.75 feet to 3.5 feet) resulted in a 5.3 percent increase in K-values. Olson et al. (*9*) noted that the difference in curve length between a 40-inch and a 42-inch driver eye height was about 3 percent.

Woods (*12, 15*) noted that stopping sight distance is relatively insensitive to changes in driver eye height. A 2.3 percent increase in curve length results from each 0.1 foot reduction in driver eye height and a 11.5 percent increase in curve length would result from 3.5 to 3.0 feet decrease in driver eye height. As shown in Figure A-3, moderate reductions in driver eye height result in small increases in vertical curve lengths; however, for large algebraic differences in grade and higher speeds, reduction in driver eye height increase vertical curve length noticeably. Thus, the additional length of curve may be quite long even though the percentage increase is small.

Object Height. The sensitivity of curve length to changes in object height has also been studied extensively. AASHTO (*1*) noted that "using object heights of less than 6 inches for stopping sight distance calculations results in considerably longer crest vertical curves" and that decreasing the object height from 6 to 0 inches increased the vertical curve length by 85 percent. Farber (*10*) determined that stopping sight distance was more sensitive to changes in object height than to changes in driver eye height. Khasnabis and Tadi (*7*) noted that a reduction in object height from 6 to 3 inches caused a 19 percent increase in the K-factor, and a reduction in object height from 3 to 0 inches caused a 61 percent increase in the K-factor. Olson et al. (*9*) also analyzed the impact of a reduction in object height from 3 to 0 inches. They concluded that a zero-inch object height requires about 10 percent more vertical curve length than present AASHTO standards.

Figure A-3 also illustrates the increase in vertical curve length that results from reduced object height values. Large increases in curve length did not occur at algebraic difference of grades of 4 percent and lower. Algebraic difference of grades of greater than 4 percent show a noticeable increase in curve length, especially when using very low object heights. Thus, it would appear that object height is more sensitive at large algebraic differences in grade and low object heights, especially around values of one inch or lower.

Woods noted that a reduction in object height ranging 6 inches to 2 inches resulted in a 3 to 4 percent increase in vertical curve length per half-inch reduction (*15*). Woods (*12*) also noted that the proposed decrease from 6 to 4 inches by Olson et al. (*9*) would increase the minimum length of crest vertical curves by 12 to 16 percent. Thus, although more sensitive than driver eye height, the effect of object height reductions on the vertical curve lengths was not as significant as expected.

Headlight Height and Angle of Divergence. The literature does not document the sensitivity of sag vertical curve length to changes in headlight height and angle of divergence. As mentioned previously, headlight heights were 2.0 feet above the pavement surface in the 1940 AASHO policy (*2*), 2.5 feet above the pavement surface in the 1954 AASHO policy (*3*), and 2.0 feet above the pavement surface in the 1965, and subsequent AASHTO policies (*4, 5, 6*). The 1.0 degree upward divergence of the light beam has not changed during this period. The two drawings at the bottom of Figure A-3 show the sensitivity of sag vertical curve length to changes in the values of these variables. Note that as the height of the beam increases, the required length of curve decreases, and as the angle of upward divergence decreases, the required length of curve increases.

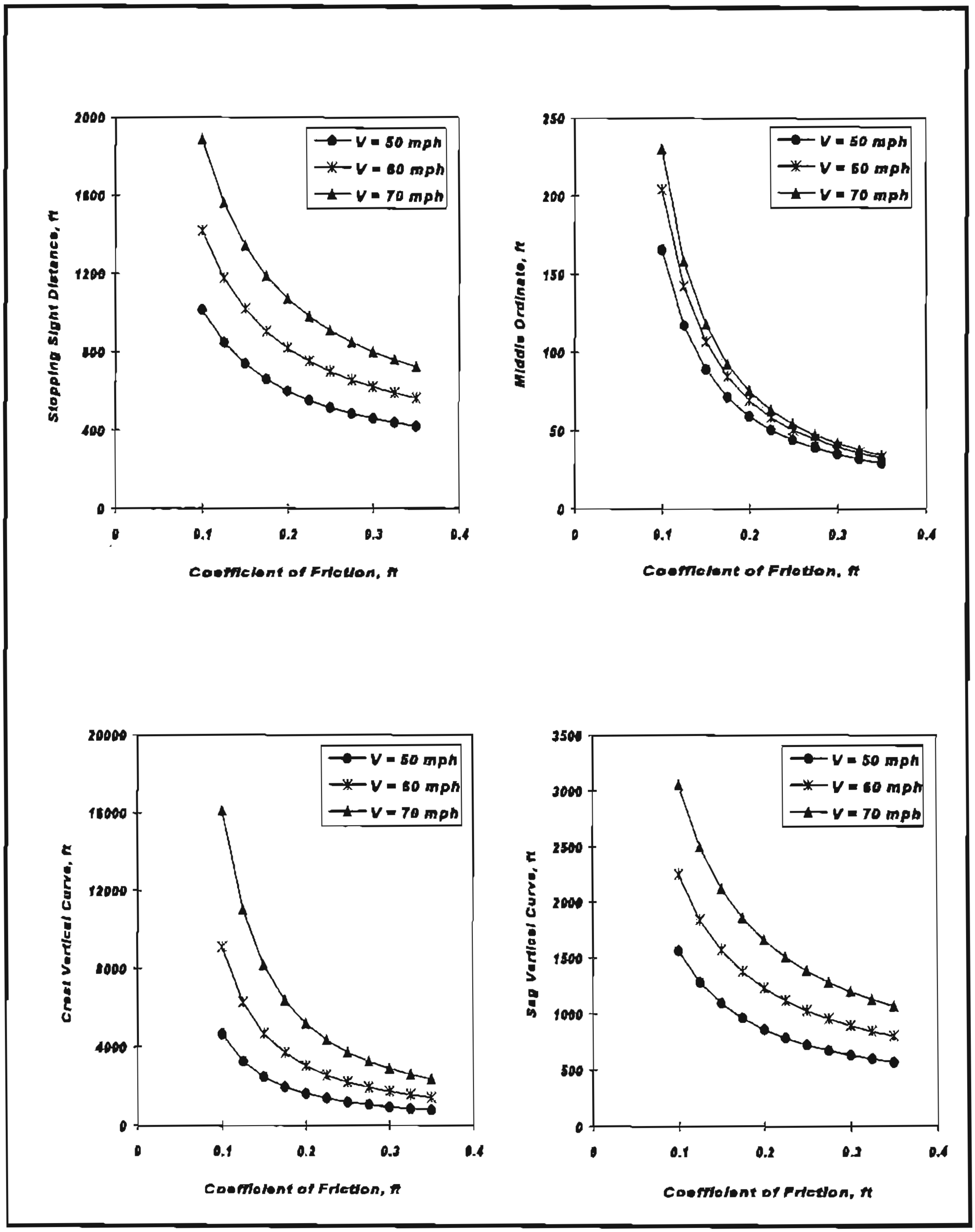


FIGURE A-2. Sensitivity of Stopping Sight Distance and Curve Geometry to Changes in Pavement Friction Values.

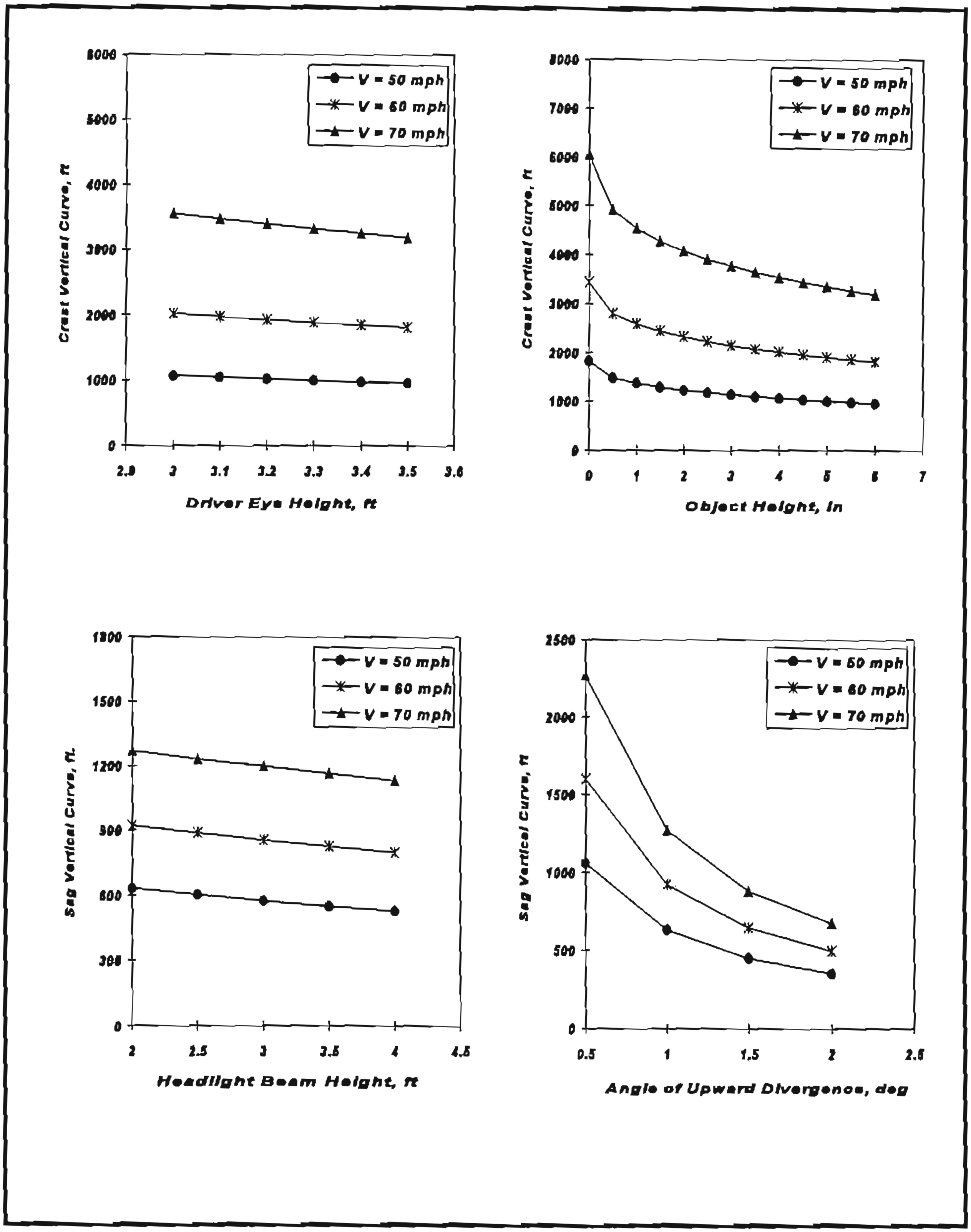


FIGURE A-3. Sensitivity of Curve Geometry Changes in Driver-Eye, Object, and Headlight Beam Heights.

Functional Analysis

Vertical curves restrict available stopping sight distance when the approach grades are steep, the vertical curve is short, or both. Current AASHTO standards (*1*) for vertical curves lengths are based on combinations of design speed and algebraic difference in the approach grades (A). The minimum and desirable lengths (L) of vertical curves produce "minimum" and "desirable" stopping sight distance at the assumed design speed.

To avoid separate tabulations for A and L, design controls for vertical curves are expressed as K-factors; i.e., the length of vertical curve to effect a 1 percent change in grade (A). Design K-factors are calculated to provide either minimum or desirable stopping sight distance at the assumed design speed. Thus, a single K-value encompasses all combinations of L and A for a single design speed, and plan sheets can be checked by comparing all curves with the design K-value.

The most important characteristics of vertical curves in reconstruction projects are the existing K-value, the available stopping sight distance, and its distribution throughout the vertical curve. A common misconception is that the minimum stopping sight distance is manifested over the entire length of the curve (*16*); however, a plot of the available stopping sight distance along the vertical curve reveals the available sight distance decreasing to a minimum value, and then rapidly increasing as the vehicle reaches the crest or low point of the curve. Such plots are called sight distance profiles (*16*). These plots can also be prepared for vertical and horizontal curves as shown in Figures A-4 and A-5 (*16*).

Sight-distance profiles are useful because they reveal the relationship between curve length, approach grade, and available stopping sight distance. The sight distance profiles shown in Figure A-4 represent crest vertical curves for different K-factors (K = 80, 120, 150, 220) and an algebraic difference in grade of 6 percent. The different K-values represent minimum and desirable stopping sight distance for design speeds of 45 and 55 mph, respectively. Horizontal lines represent minimum and desirable stopping sight distances for a design speed of 55 mph. Thus, when the available stopping sight distance curve falls below one of the horizontal lines, stopping sight distance is less than the AASHTO criteria for a 55 mph design speed.

Inspection of the sight-distance profiles illustrated in Figure A-4 reveals three basic characteristics of available stopping sight distance at vertical curves (*16*):

1. Vertical curves that create limited stopping sight distance do so over relatively short lengths of highway. Similarly, less severe stopping sight distance limitations (higher K-values) affect longer sections of highway;

2. The length of highway over which stopping sight distance is at a minimum is relatively short compared with the length of a vertical curve; and

3. For a constant K-factor, the length of highway over which stopping sight distance is limited increases as the algebraic difference in grade increases; however, the minimum available sight distance remains the same.

Figure A-6 shows this last observation more clearly. This figure illustrates the length of roadway with stopping sight distance less than 450 feet (minimum stopping sight distance for 55 mph) as a function of crest curve geometry. The K-values of 50, 60, 80, and 120 correspond to minimum stopping sight distances of 250, 275, 325, and 400 feet (design speeds of 30, 35, 40 and 45 mph, respectively). Note that for any given K-value, the length of roadway with available stopping sight distance less than 450 feet increases as the algebraic difference in grades increases. In addition, the closer the minimum available sight distance is to 450 feet, the longer the length of roadway with limited stopping sight distance.

Inspection of the sight-distance profiles in Figure A-5 reveals two basic characteristics of available stopping sight distance at horizontal curves (*16*):

1. The sight distance restriction is usually unidirectional; however, for very extreme restrictions, it differs in the direction of travel between the inner lane and second, or outer lane. Generally, only vehicles traveling in the inside lane are subjected to the greatest restriction. Vehicles in the outside lane have an additional lane (12 feet) of lateral offset, which increases available stopping sight distance for these vehicles; and

2. Sometimes (i.e., near vertical obstructions caused by retaining walls, rock cuts, buildings, or rows of trees), driver eye and object heights are not factors in determining available stopping sight distance.

Summary

The determination of required stopping sight distances is based on the distance required to react to a hazard and bring a vehicle to an emergency stop, and as a minimum, making that length of roadway visible to the driver. The preceding discussion has described the historical development of the AASHTO equations for determining stopping sight distances and vertical and horizontal curve lengths, the sensitivity of these equations to changes in the parameters within the models, and a functional analysis of available stopping sight distance for a variety of curve geometries. The AASHTO equations were first published in the 1940s and, except for modifications to individual parameters, have not changed since that time. Design for all types of roadways use these same model and parameter values.

The sensitivity analysis showed that stopping sight distance is most sensitive to changes in the coefficient of friction; however, assumed coefficients of friction are conservative because they represent wet pavements and worn

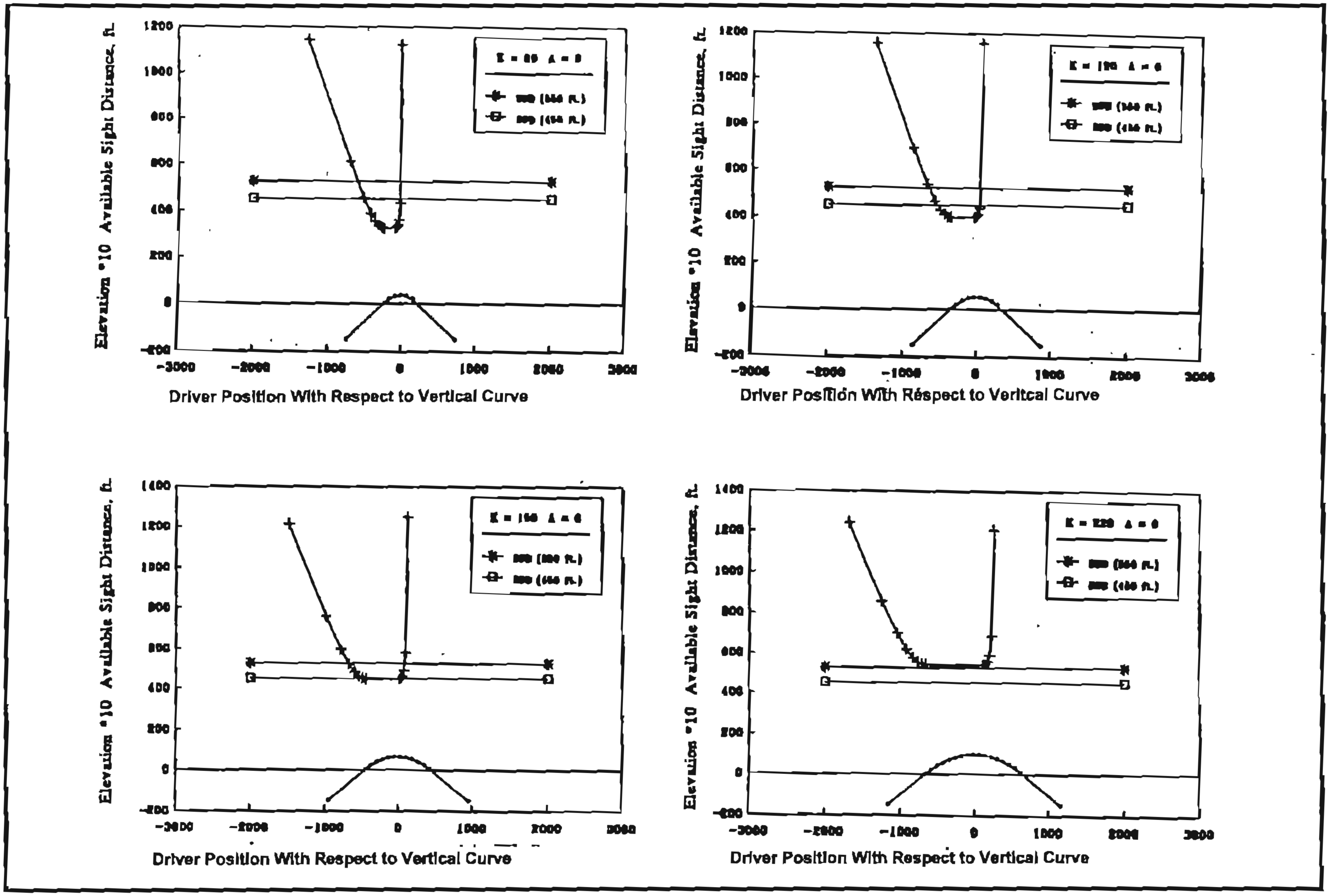


FIGURE A-4. Available Sight Distances as a Function of Curve Geometry, A=6.

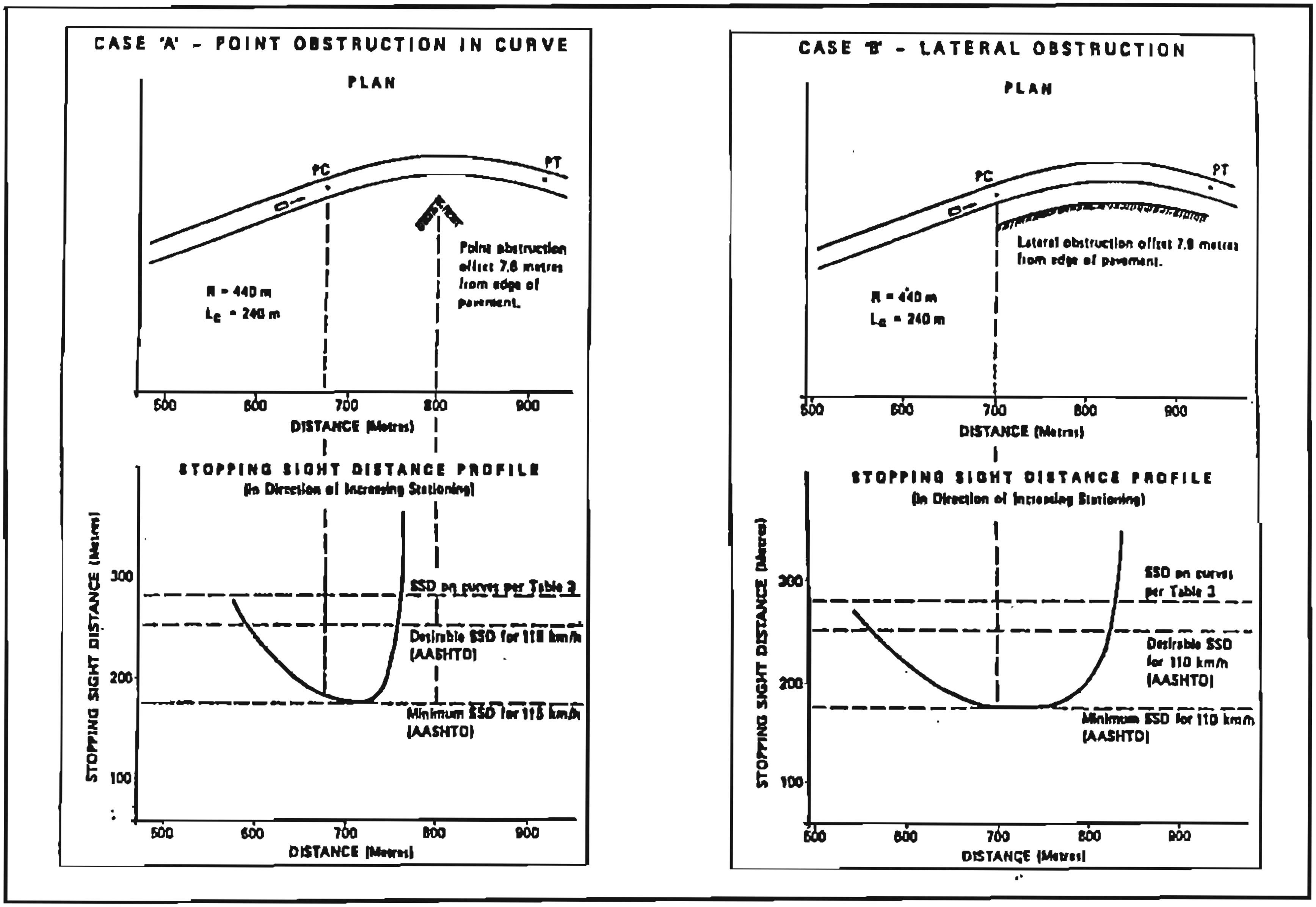


FIGURE A-5. Stopping Sight Distance Profiles for Horizontal Curves.

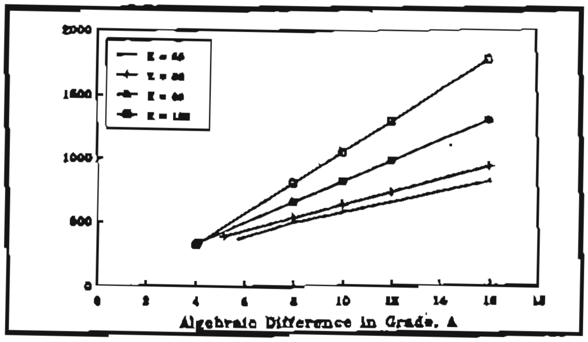


FIGURE A-6. Length of Roadway with Stopping Sight Distance Less than 450 Feet.

tires. A question arises related to different coefficients of friction for different types of roadways. Lower volume roads may not be built or maintained to the standards of higher volume roads; however, they may have higher friction values because of lower traffic volumes. Higher volume roads are built and maintained at higher standards, but have more traffic to wear down the surface.

Stopping sight distance is also sensitive to changes in perception-reaction time. Some researchers believe that the perception-reaction time should be longer to include all potential situations, while other researchers feel it should be shorter. Some researchers believe that perception-reaction time may vary according to type of roadway. The "Green Book" (*1*) notes that drivers on urban facilities confronted by possible conflicts with crossing vehicles may be more alert than the same driver on a limited access facility; however, the driver on the lower classification road also may be distracted by adjacent roadside developments, whereas the driver on the limited access facility may be more attentive due to interaction with other traffic.

The driver eye height is the parameter that least affects vertical curve length and the object height is only slightly more influential. The driver eye height has changed three times since the equation was first adopted in 1940 and object height has changed twice. The current value for driver eye height is generally well accepted and it seems reasonable that drivers do not vary according to roadway types; however, objects may vary depending on the type of roadway. It also seems reasonable that the object height for the stopping sight distance model should reflect hazardous objects that drivers are likely to encounter on different types of roadways.

Vertical and horizontal curves that create severe stopping sight distance limitations do so over relatively short sections of highway, and curves that create less severe stopping sight distance limitations do so over longer sections of highways. Some accident studies have shown that more accidents occur on sections with less severe stopping sight distance limitations (longer horizontal or vertical curves) than on those with more severe limitations (shorter horizontal or vertical curves). This contradiction could be due to the time and distance that the vehicle and driver are exposed to the sight limitation. The severe sections are relatively short and the segment is passed quickly. Less severe sections are usually longer. Thus, the driver has a greater opportunity to encounter a potentially hazardous situation. It should be noted that in both cases, adequate sight distance is available for stopping on dry pavement. This observation might partially explain why so few accidents occur at limited sight distance locations.

ALTERNATIVE STOPPING SIGHT DISTANCE MODELS

Stopping sight distance is one of the most important considerations when designing a highway because it is required at all points along the roadway; however, there is continuing and growing concern regarding the current policy and the general approach to determining stopping sight distance (*17*). Currently, the AASHTO model uses the same parameter values for all functional classes of roads. The equation for stopping sight distance uses four variables: speed, perception-reaction time, friction, and grade. Object height and driver eye height are related to the stopping sight distance through the crest vertical curve lengths. The parameters within the model have been studied extensively, but the model itself has not been questioned. The following sections discuss alternative stopping distance models described in the literature.

Functional Classification Model

Neuman (*17*) presented an approach to stopping sight distance design in his paper that involved abandoning the idea that a single operational modeis appropriate for all highway types under all conditions. Instead, he suggested functional highway classification as the foundation for determining stopping sight distance design policy and values. Different driver, vehicle, and roadway parameter values for different classes of roadways would result in a range of stopping sight distance values for a given design speed, rather than just one value for all conditions.

Neuman's functional classification model considers critical events that might serve as the basis for stopping sight distance rather than the relatively infrequent conflict with a 6-inch object. It also recognizes the drivers' expectations for the type of trip and the complexity of the driving task along a specific highway functional type. Neuman offered four key elements to support the use of functional highway classification to determine required stopping sight distances.

1. Stopping sight distance requirements are considered related to many possible operational events, rather than only one event.

2. Inherent differences are present in the operating characteristics and safety experiences of different highway types. The overall design approach should recognize these differences by using models that relate to each highway type.

3. Human factors and vehicle-roadway parameters also differ with roadway type.

4. Stopping sight distance requirements differ along the same highway. Demands on drivers and vehicles and probabilities that critical operations will occur are not uniform, but vary according to other physical, geometric, and operating conditions.

For each highway type, many critical events and parameter values could serve as the basis for an operational stopping sight distance model. Table A-4 summarizes the values proposed by Neuman (*17*). The table also illustrates the stopping sight distance values that would result for the particular highway type. Neuman noted that the exact values cannot be fixed, but should be tested through further research. He provided example parameter and design values to illustrate model concepts and show the sensitivities that should be a part of stopping sight distance policy.

Neuman concluded that if the functional classification approach were adopted for highway design, greater flexibility would exist within the presentation of standard values. The result would be more cost-effective designs that provide additional sight distance where most needed. Such cost-effectiveness would be achieved within the model's framework, rather than through design exceptions; however, it should be noted that the parameter values proposed by Neuman have not been validated with field data, and typically, are not consistent with research results identified by this study.

Driver Expectancy Models

Messer et al. (*18*) presented a model for measuring driver workload that is an indirect measure of driver expectancy. The basic premise of his model is that changing the drivers' built up expectancy by suddenly increasing their workload (violating their expectancy), means that they are more likely to make a mistake (have an accident). Design consistency models evaluate the speed differentials related to the driver expectancy. Design consistency and driver workload models are discussed in the next sections.

Design Consistency Models. Several methodologies have been developed for analyzing speed differential along a section of roadway. These methodologies generally focus on reducing or limiting the magnitude of the speed differential between geometric elements. Additional attention is placed on potential discrepancies between the design speed used for specific geometric elements (design speed) and the speed at which motorists actually drive (operating speed). Several researchers have addressed specific effects of tangents and horizontal curves on driver speed changes and expectations. Although differing in specific requirements, the common objective of the design consistency procedures is to provide a consistent highway design without requiring abrupt speed changes that might surprise the driver.

Research has focused on ways of providing a consistent speed environment that conforms to driver expectancies, and does not require abrupt changes in operating speed. Several different design consistency procedures have been presented in the literature. One of the simpler models was presented by Lamm et al. (*19*). This procedure concentrates on operating speed changes induced by horizontal curvature and tangent length, while also examining the change in degree of curvature of horizontal curves along the roadway. Lamms' procedure focuses on achieving a consistent horizontal alignment by minimizing abrupt changes in operating speed and keeping the change in degree of curvature to a minimum. Roads are rated as good, fair, or poor as shown in Table A-5.

One of the most interesting concepts in Lamms' procedure is the classification of tangents as "independent" or "non-independent." A tangent is classified as independent when the tangent length is long enough for vehicles to reach 85th percentile speeds of 58 mph. A chart presents lengths of tangents coupled with the 85th percentile speed in succeeding curves, enabling the user to predict whether the tangent should be considered as independent or non-independent. This approach to deciding the basis for which features should be compared, i.e., curve-to-curve to tangent-to-curve, appears promising for analyzing feature sequences; however, it is limited in that it ignores any changes in speed related to vertical alignment, and neglects the effects of other roadway features.

A procedure introduced by Leisch and Leisch (*20*) includes the influence of both horizontal curvature and vertical grades. They suggested that variation in passenger car speeds of more than 10 mph, reductions in design speed by more than 10 mph, and differences in speed between trucks and passenger cars of more than 10 mph should be avoided. The objective of their procedure is to enable the designer to identify areas of the highway alignment that violate these recommendations. Truck speeds are predicted from tabular values presented in the AASHO 1965 *A Policy on Geometric Design of Highways and Streets* (*4*), while speeds of automobiles are determined using equations derived from driver characteristics. The main limitation to the approach is that it ignores changes in the roadway environment, other than those of vertical and horizontal alignment. For example, intersections, bridges, shoulder widths, etc., are ignored by the process.

Great Britain uses a "traffic speed" concept to examine proposed highway alignments (*21*). The traffic speed is determined through the examination of observed speeds on adjacent sections of roadway and the analysis of the impacts of proposed geometric design elements. The consistency of the proposed roadway elements is compared using the results of the analysis. Sweden has taken this process a step further, and uses a computer program to create a predicted speed profile along proposed roadway alignments. They then study speed profiles to enable the comparison of alternate alignments and the effects on speed consistency.

Switzerland uses both a design speed and a project speed for arriving at proposed highway alignments (*21*). The design speed used is similar to AASHTO guidelines (*1*) in

TABLE A-4. Critical Events and Parameter Values for an Operational Stopping Sight Distance Model.

	Object Height	Perception- Reaction Time (sec)	Coefficient of Friction	Stopping Sight Distance (ft)
Two-Lane Low-Volume Road	single-vehicle encounter with a large object	1.5	0.40 at 30 mph 0.34 at 50 mph	141 at 30 mph 507 at 50 mph
Two-Lane Primary Rural Highway	vehicle-vehicle conflict involving crossing or stopped vehicle	3.0	0.30 at 30 mph 0.28 at 50 mph	343 at 40 mph 891 at 70 mph
Multilane Urban Arterial	vehicle-vehicle rear-end conflict	2.5	0.37 at 30 mph 0.31 at 50 mph	189 at 30 mph 452 at 50 mph
Rural Freeway	single-vehicle conflict with small (0 to 6-in) object	2.5	0.28 at 50 mph 0.25 at 70 mph	518 at 50 mph 989 at 70 mph
Rural Freeway	vehicle-vehicle conflict (rear-end)	3.0	0.22 at 60 mph 0.20 at 80 mph	545 at 60 mph 1,074 at 70 mph
AASHTO	vehicle-object conflict (6 in)	2.5	0.40 at 20 mph 0.28 at 70 mph	125 at 20 mph 850 at 70 mph

TABLE A-5. Measure of Horizontal Alignment Consistency.

	Good	Fair	Poor
Range of Change in Degree of Curve	$\Delta DC \leq 5$	$5 < \Delta DC \leq 10$	$\Delta DC > 10$
Range of Change in Operating Speed	$V_{85} \leq 6$	$6 < V_{85} \leq 12$	$V_{85} > 12$

that it provides a minimum design value for various roadway features (i.e., sight distance, horizontal curvature, etc.), while the project speed is the "maximum speed expected in a certain roadway section and serves as a test speed to assess adequate sight distances, adequate radii of crest or sag vertical curves..." Switzerland uses a speed prediction model to examine the horizontal roadway alignment, predicting project speeds throughout the alignment. The examination of changes in project speed may detect abrupt changes in speed and speed transitions along the roadway. Limiting maximum changes in operating speeds and speed changes between adjacent roadway elements ensures design consistency.

Germany also uses a design speed and an operating speed in roadway design (*21*). Operating speed corresponds to the predicted 85th-percentile speed on a facility. An acceptable alignment would have a predicted operating speed that does not exceed the design speed by more than 20 km/h. German designers also use alignment changes to provide speed transition sections when passing from high-speed rural areas to low-speed populated areas, i.e., introducing curvature that might otherwise be unnecessary. Speed transitions are controlled by examining a "curvature change rate" to ensure that transitions are gradual and safe between adjacent roadway sections. Other checks on horizontal alignment include controls on successive curves, tangent lengths, and the number and severity of curves along stretches of roadway. Mandating that "the length of tangent (in meters) between curves cannot exceed twenty times the design speed of that roadway (in km/h)" ensures a curvilinear environment.

Design practices regarding horizontal alignment in France are to provide "...safety, consistency of the alignment as measured through the speed profile, and comfort" (*21*). Tangent length restrictions are enforced with maximum tangent lengths ranging from 2 to 3 kilometers and tangent sections limited to 40 to 60 percent of the total length of long roadway sections. These restrictions were developed to avoid problems with driver fatigue.

Research in Australia has focused on the differences between design speeds and desired or operating speeds (*22*). Desired speed is defined as the 85th percentile speed measured on tangent sections of roadway within a particular roadway section. For high-speed alignments, Australian practice is to continue to provide the conservative design features provided previously, since this practice is consistent with driver expectation; however, for low-speed alignments, extensive changes have been outlined. These changes are as follows:

- The design speed will be the predicted 85th percentile speed;
- The limiting values for design criteria directly related to driver speed behavior should be consistent with the behavior of the 85th percentile driver; and
- Where there is no evidence that existing geometric standards are inappropriate, they should be retained.

In other words, standards for lower-speed alignments have been adjusted to match actual driver behavior. Following the preliminary selection of horizontal curve radii, projected 85th percentile speeds are estimated for the curves. Those speeds are then used to specify other design parameters.

Driver Workload Models. All of the methodologies discussed so far have concentrated on treating roadways so that the *observed* driver responses (i.e., driver speed changes) conform to specific ranges determined to be acceptable. A premise implicit in those methodologies is that drivers can observe and analyze the roadway ahead to arrive at an appropriate speed. If drivers drive too fast on sharp curves that follow tangents, observing the speed changes between the two features will result in an underestimation of the design consistency. Watts and Quimby (*23*) established that drivers do not always recognize the risk potential of roadway features—questioning procedures that rely strictly on the results of driver-controlled speed changes as reasonable. Further examination of consistency and expectancy leads to the workload concept that more directly addresses the influence of the roadway on the driver.

The driving task imposes work on the driver, and this workload varies greatly in difficulty and frequency. The effect of workload on driver performance may be affected by driver expectations and driver capabilities. Roadways that present inconsistencies in their design would be expected to violate driver expectancies and impose higher workloads on drivers. Senders (*24*) defined workload as "a measure of the effort expended by a human operator while performing a task, independently of the performance of the task itself." Knowles (*25*) defined workload as the answer to two questions: "How much attention is required?" and "How well will the operator be able to perform additional tasks?" The definition presented by Knowles seems appropriate to the driving environment, because it consists of many overlapping tasks, each requiring the driver's attention. AASHTO (*1*) notes that "A common characteristic of many high-accident locations is that they place large or unusual demands on the information-processing capabilities of drivers."

When examining the effects of workload on human performance, the Yerkes-Dodson law (*26*) offers some insight regarding driver performance at various levels of arousal. As illustrated in Figure A-7, the Yerkes-Dodson law states that quality of performance increases as arousal level increases until some peak performance level is reached; however, after that point, the performance level decreases with further increases in arousal level. Representing arousal as workload, the Yerkes-Dodson law suggests that performance level is at a peak for some optimum value of workload, and is at a lower level for workloads less than and greater than the optimum workload.

The shape and position of the response curve varies according to the nature of the tasks, with simple tasks resulting in higher quality performance than more complex tasks; .i.e. simple tasks do not suffer from as much variation in performance quality as arousal levels change. The translation of these ideas to driver workload seems to suggest that roadways with relatively complex designs and many features in close proximity (high workloads) would result in driver performance that would be degraded below some optimum driver performance. Likewise, roadways with few features and very low workloads would also exhibit degraded driver performance.

Kahneman (*27*) interpreted the increased likelihood of failure at lower levels of arousal as the result of motivational factors. The subject either fails to accept the task, or fails to adequately evaluate their performance. The probable cause of failure at high levels of arousal is that the subject fails to attend to relevant cues necessary for optimum performance. As the arousal level increases, irrelevant information is screened out; however, at some point, useful information is also screened out as the subject attempts to limit incoming information to a level he or she can process.

Peripheral Effects. One effect of increasing mental workload was examined by Acosta and Dickman (*28*). They studied the effect of increased mental workload on processing visual information presented in test subjects' peripheral areas. They determined that increased mental workload results in a narrowed visual field. Although the work was not conclusive in nature, the findings were nevertheless important in implication. Locations that have a high workload are usually

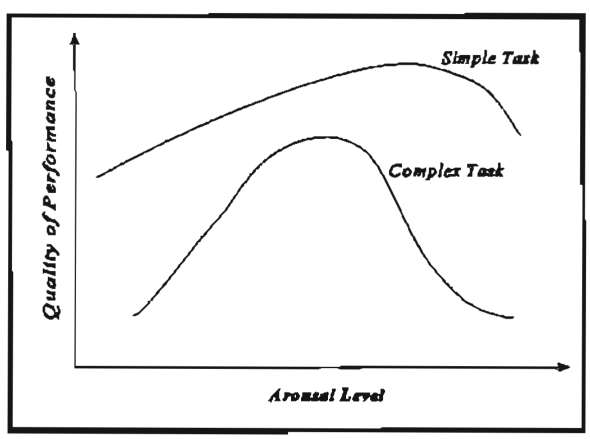


FIGURE A-7. Yerkes-Dodson Law.

those with a high potential for conflict with objects or other vehicles. If the driver does not see an object (e.g., a vehicle or an animal) in their peripheral vision, they probably cannot avoid striking that object. An accompanying phenomenon that increases the opportunity for accident occurrence is that response times to auditory stimuli increases as the number of stimuli increases (*29*). Clearly, areas of high workload could result in a higher accident risk due to delayed driver response.

Conceptually, one would expect that a driver traveling on a straight or nearly straight section of roadway with a relatively low workload would be more likely to make a mistake when suddenly encountering a feature or features that impose a high workload. This situation could be contrasted with that in which a driver traveling on a winding section of roadway with a relatively high workload encounters a feature that imposes a high workload. Generally, greater accident risks are associated with high workload features; however, the distinction between the two situations seems clear in that the feature that presents the driver with a sudden increase in workload would probably have the higher accident rate..

In a study examining the relationship between roadway environment and curve accidents, Matthews and Barnes (*30*) examined the relative accident risks of short radius curves that were found after long and relatively straight sections of roadways. In each case, the accident risk on these curves was approximately four to seven times higher than the accident risks on standard curves. Although their study did not examine workloads, the analogy between the previously stated hypothesis and their findings seems clear—drivers that are traveling along a relatively straight path with a low workload and are suddenly presented with a situation that demands that the driver respond correctly to changes in the environment are more likely to make mistakes. The drivers' performance may decline, and an accident may result due to the sudden increase in a workload.

Messer Procedure. Messer developed a driver workload model by gathering empirical data regarding driver expectations of roadway features and relating violations of those expectancies to workload (*18*). His methodology is based on the presumption that the roadway itself provides most of the information that the driver uses to control his or her vehicle, and that the roadway imposes a workload on the driver. Workload is higher during encounters with complex geometric features and can be even higher when drivers are surprised by encounters with unexpected or unusual geometric features. Messer's model quantifies design consistency by computing a value for driver workload. The technique relies on a set of assigned ratings for various roadway elements—bridges, divided highway transitions, lane drops, intersections, railroad grade crossings, shoulder-width changes, alignment, lane-width reductions, and the presence of crossroad overpasses. The ratings are based on the type and severity of the design element, and modified according to their location. Influencing factors include sight distance to the element, similarity to previous elements, workload of previous segments, and percentage of familiar drivers.

The workload along the roadway is estimated using an equation that defines a subjective level of consistency (LOC) in terms related to driver workload. The methodology is applicable to two or four-lane highways in flat or rolling terrain, and may be used to examine existing or proposed highways. The equation used for calculating the driver workload is shown as follows:

$$WL_n = (U \times E \times S \times R_f) + (C \times WL_l) \qquad [8]$$

where: WL_n = driver workload;
U = driver familiarity factor;
E = feature expectancy factor;
S = sight distance factor;
R_f = basic workload potential rating;
C = carryover factor; and
WL_l = workload of the previous feature.

The combination of empirical data and the experience of a group of experts (*18*) defined the range of values for each of these terms.

Sample Application of Various Consistency Measures. Several different consistency and workload measures have been presented in the literature. Although a comparison of procedures on one roadway does not completely address differences and similarities between the various procedures, such a comparison may be of interest. Lamm et al. (*31*) present a hypothetical alignment (Figure A-8) for which three different speed consistency analyses have been conducted. McLean (*22*) has used the same hypothetical alignment to evaluate an Australian methodology. A fifth measure of consistency, based on workload (*18*), has been added through the application of Messers' procedure. The results of this comparison are shown in Table A-6.

Note the large variation in predicted speeds resulting from the four design consistency procedures. The Leisch methodology (*20*) generally predicts the lowest speeds, while the Australian (NAASRA) methodology (*22*) predicts the highest speeds. One explanation for the differences in predicted speed may lie in differences in study location and design practices, i.e. differing cultures and environments may explain much of the variation in speeds. Leisch and Leisch

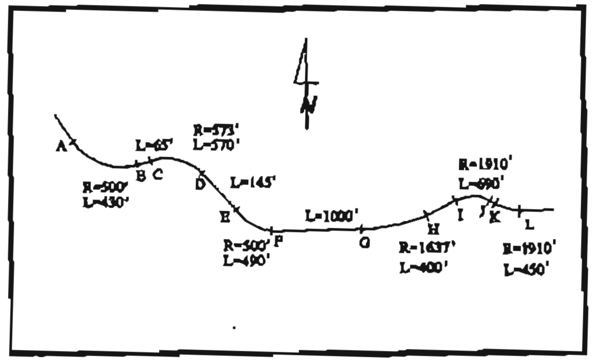


FIGURE A-8. Hypothetical Alignment (*20, 31*).

TABLE A-6. Comparison of Operating Speed Predictions (km/h).

Method	Curve					
	AB	CD	EF	GH	IJ	JK
Leisch W-E	60[1]	63	60	71	76	85
Leisch E-W	60	63	60[1]	93	97	97
Swiss	69[1]	69[1]	69	100	100	100
German	70[1]	70[1]	70	86	86	86
NAASRA W-E	85	77	76	91	100	100
NAASRA E-W	76[1]	77	82[1]	99	100	100
Messer W-E	F[2]	F[2]	F[2]	B	B	B
Messer E-W	E	F[2]	F[2]	E	D	B

[1]Unacceptable because of speed consistency criterion.
[2]Unacceptable because of high workload.

based their recommendations on American drivers, the Swiss and Germans on their respective compatriots, and NAASRA on Australian drivers. Still, it is significant that all four operating speed checks identified the same two curves (AB and EF) as problem locations. This agreement shows similarity of results, suggesting a convergence of research and methodology.

A fifth measure of consistency provided by the application of the Messer procedure (*18*), is very different in implication when applied to Lamms' hypothetical alignment (*31*). The results from the Messer procedure are reported in a range extending from "no problem expected" to "big problem possible." In addition to curves AB and EF, the Messer procedure predicts that problems may occur for curve CD for the W-E direction of travel, and to a lesser extent, for curves CD, GH, and IJ. The Messer procedure was applied using assumptions of a 140-foot right-of-way, sight restrictions at the limits of that right-of-way, and the speeds predicted by Leisch and Leisch (*20*) as representative of the 85th percentile speeds on the alignment.

Significant differences exist in the application of the various design consistency procedures. Although further information regarding the hypothetical alignment (e.g., providing a vertical alignment, showing structures, etc.) would provide a better comparison of the methodologies, this simple example provides a basis for comparison. The Messer procedure is highly sensitive to horizontal curvature; this sensitivity generally accounts for the differences in results. The speed profile procedures assume that once a driver has slowed for one curve, similar curves are of little consequence. Messer's workload procedure assumes that combining high-workload features in close proximity results in a higher workload for the subsequent geometric features.

Workload and Safety. Statistical analyses were performed on geometric data and the accident histories for 21 restricted sight distance highways in central Texas (*32, 33*) to investigate whether a relationship exists between driver workload and safety. The analysis focused on the relationship between accidents and mean driver workload, and accidents and variance (yaw) in driver workload. Because available sight distance plays a major role in a drivers' workload, the results illustrated in Figures A-9 and A-10 are particularly relevant to this study.

Note that slight to moderate increases in driver workload do not seem to affect accident rates; however, large increases in driver workload (i.e., $EWL_B > 6$) are associated with higher accident rates. These findings are consistent with previous studies (*34*) of accidents and limited sight distance. Limited sight distance by itself (low driver workload) does not significantly affect accident rates; however limited sight distance in combination with other geometric features (high driver workloads) results in increased accident rates. For example, placing an intersection within a limited sight distance section of roadway may create a safety problem because of limited intersection sight distance.

Probabilistic Models

Designers assume that when the published geometric design standards are properly applied, roadways have been designed with an appropriate margin of safety; however, this assumption may not always be correct. Navin (*35*) proposed a probabilistic method for calculating meaningful measures of road safety. A basic assumption was that all parameters are independent, normally distributed random variables. Current design parameters were used with expected values to

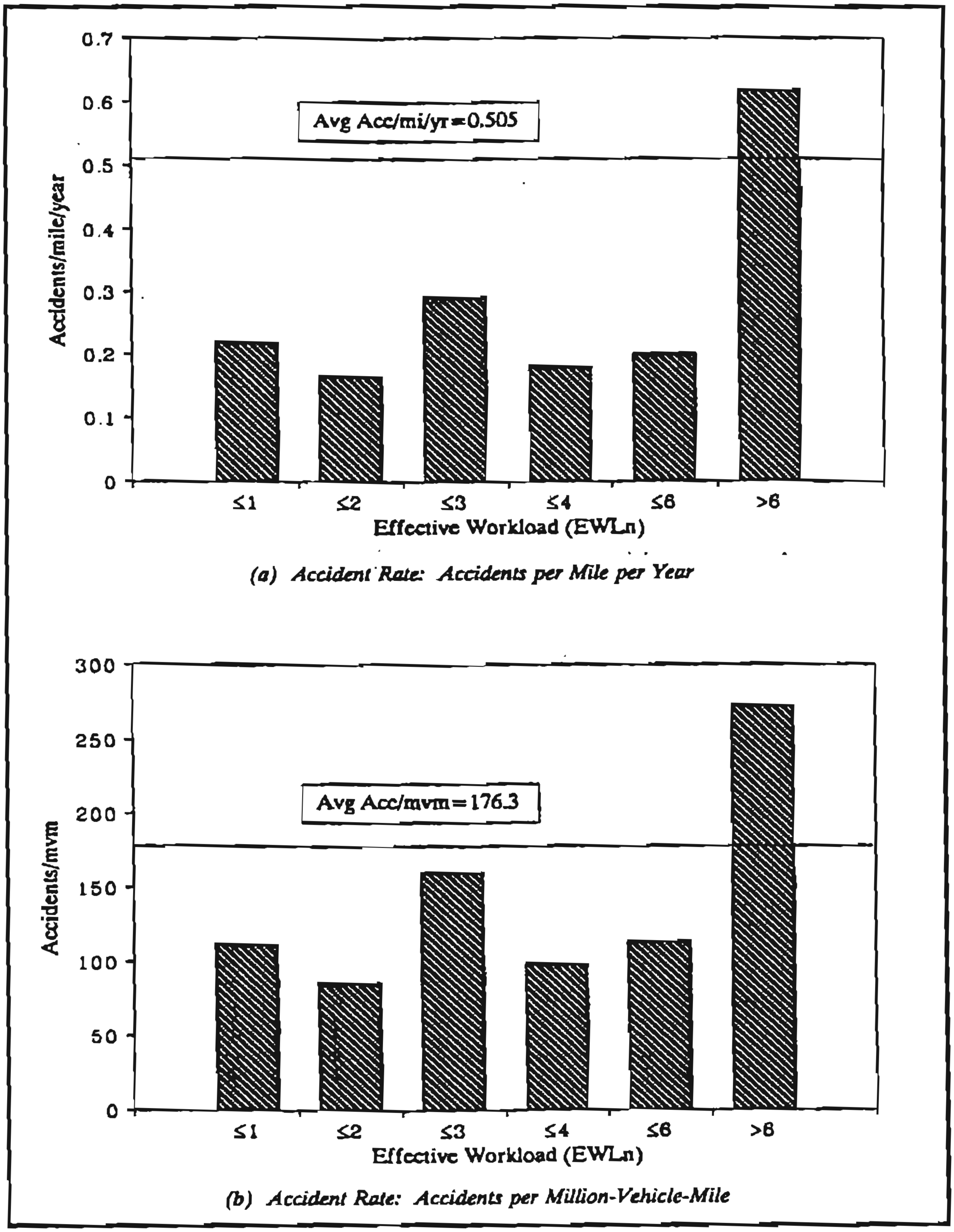
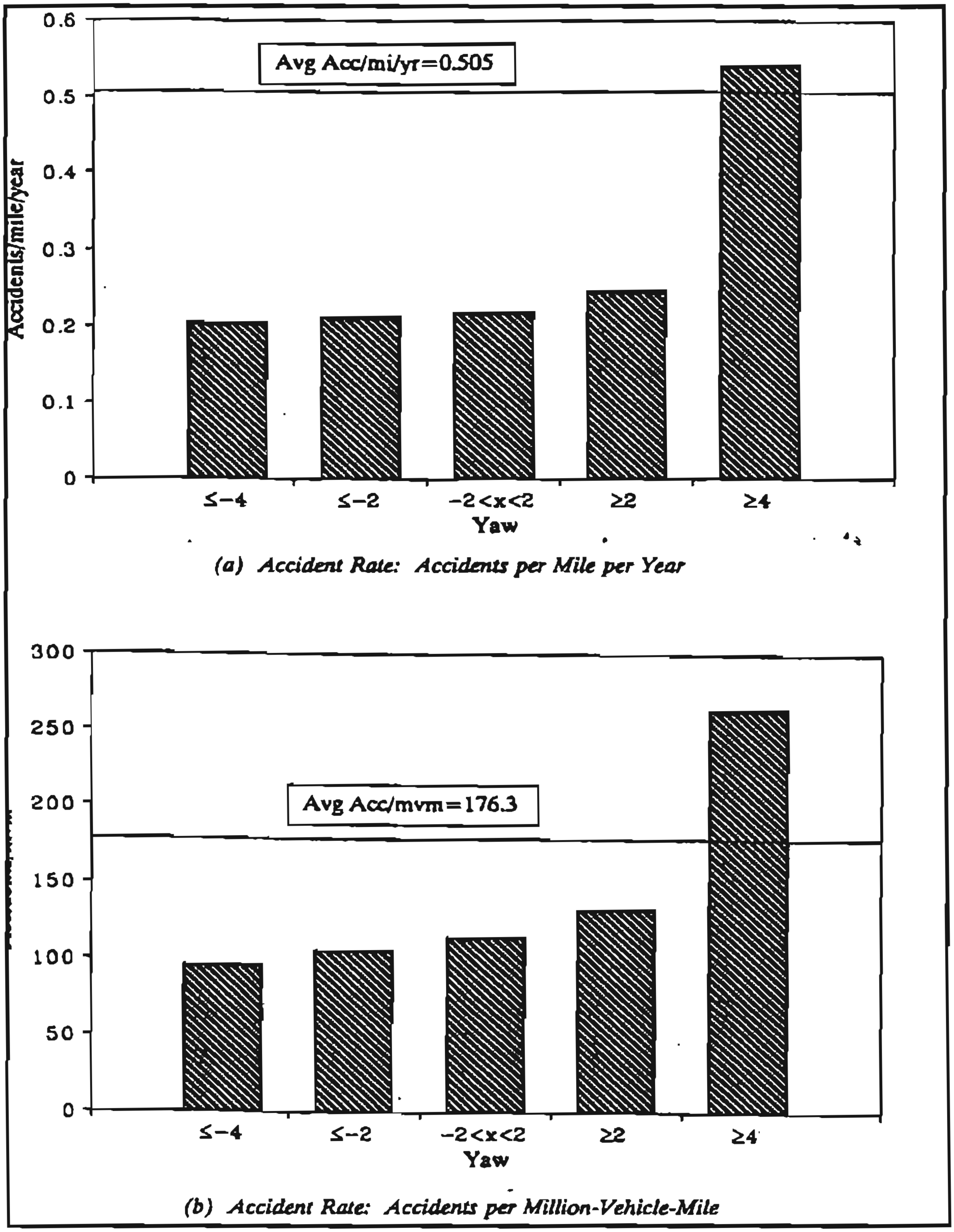


FIGURE A-9. Accident Rates for Various Levels of Driver Workload.



(a) Accident Rate: Accidents per Mile per Year

(b) Accident Rate: Accidents per Million-Vehicle-Mile

FIGURE A-10. Accident Rates for Various Driver Workload Yaw Levels.

calculate estimates of the mean and variance of these parameters.

Navin defined demand as the driver-vehicle needs for safe travel and supply as the parameter values provided by the current standards. He defined failure as demand exceeding supply; however, it should be noted that such failure does not necessarily result in an accident. The factor of safety allowing for uncertainty about exact values can be expressed as follows:

$$SF = \frac{S_e - k\sigma_{S_e}}{D + k\sigma_{D_e}} \qquad [9]$$

where:
- SF = safety factor;
- S_0 = average supply;
- k = multiple of the standard deviation (σ);
- D = average demand;
- σ_{S_0} = standard deviation of supply; and
- σ_{D_0} = standard deviation of demand.

A measure of the margin of safety is the difference between the expected value of the supply and the expected value of the demand. The ratio of the margin of safety and the combined variance is called the reliability or safety index. The probability of failure given by the stopping sight distance safety index must be evaluated using statistical methods (*36*) because the equation is not a linear combination of random variables.

Navin's procedure was applied to the SSD model and the parameter values for the minimum and desirable stopping sight distance values. Comparisons were made between the stopping sight distance supplied by the highway system and the stopping sight distance demanded by the driver-vehicle system. The mean and standard deviation for the stopping sight distance parameters were obtained from AASHTO (*1*), Olson (*9*), and Navin (*37*). Results were obtained for minimum and desirable criteria for a speed of 60 mph (*35*). The findings were as follows:

- Desirable values—stopping sight distance is 650 feet, margin of safety is 200 feet, safety index is 1.2, and the chance of failure is about 1 in 10.
- Minimum values: stopping sight distance is 525 feet, margin of safety is 75 feet, safety index is 0.4, and the chance of failure is about 3 in 10.

It should be noted that failure has been defined as the driver-vehicle system demanding longer stopping sight distances than provided by the highway design; however, as previously mentioned, such a failure will result in an accident only if the particular circumstances exist (*35*).

Much additional research is required to effectively use probabilistic methods. First, information on performance limits for normal operations, human tolerance, and vehicular limits are needed. The statistical nature and interaction of the parameters related to stopping sight distance and road geometry also are necessary. In addition, the appropriate parameter values, margin of safety, and safety index must be determined. Finally, the information must be compared with relative safety levels associated with different vehicle populations.

International Models

Harwood et al. reviewed international sight distance practices and concluded that most countries use design criteria based on similar stopping sight distance models (*38*). The countries included in this review were Australia, Austria, Canada, France Germany, Great Britain, Greece, South Africa, Sweden, Switzerland, and the United States. Many similarities exist between international design policies for stopping sight distance; however, differences exist in specific assumptions used in some countries as well as their philosophy and approach. Each country has unique insights from which geometric design researchers and policy makers could benefit greatly.

Most countries emphasize consistency between design elements. Legal principles in Europe hold the driver ultimately responsible for accidents, thus, the design community uses geometric criteria as guidelines, not standards. This practice allows designers to modify new designs on a case-by-case basis without fear of being held liable for substandard design practice at a later stage. Most national organizations work with the research community, local authorities, and consulting engineers to develop design guidelines. This promotes a faster and more direct transfer of research into practice. In 1984, Federal Highway Administration representatives traveled to Europe to broaden U.S. understanding of European practices in three highly visible highway topical areas (*21*), one including geometric design. In 1992, another researcher visited five European countries to review their geometric design practices (*39*).

$$SSD = 0.278\, V_e t + \frac{(0.278)^2}{g} \int_V^0 \frac{V}{f_T(V) + G/100 + F_L/mg} \qquad [10]$$

Three countries (Austria, Germany, and Greece) use a stopping sight distance model that incorporates the effect of a speed-dependent longitudinal friction factor and the aerodynamic drag on the decelerating vehicle. This model uses the same term for brake reaction distance as Equation (1), but uses a modified term for deceleration distance:

where:
- g = acceleration of gravity (9.81 m/sec^2);
- V = speed at any point in the deceleration maneuver (km/h);
- $f_T(V)$ = speed-dependent longitudinal friction factor;
- F_L = aerodynamic drag force (N); and
- m = mass of vehicle (kg).

The aerodynamic drag force is determined as:

$$F_L = 0.5\ \gamma\ C_w\ A\ (0.278V)^2 \qquad [11]$$

where: γ = density of air (1.15 kg/m³);
C_w = aerodynamic drag coefficient;
A = projected frontal area (m²);

The term $f_T(V)$ represents the variation of the braking friction coefficient as a function of speed, a concept originally developed by Lamm (*40*). The Austrian, German, and Greek equations used for $f_T(V)$ are included in the discussion of each country's stopping sight distance design policy.

Australia. The National Association of Australian State Road Authorities (NAASRA) is in charge of the design policies for Australian highways (*41*). NAASRA defines sight distance as the distance a vehicle will travel before coming to rest under hard braking after first seeing a hazard in the roadway. It is calculated using a reaction time of 2.5 seconds, longitudinal friction factor of 0.5, and an approximate operating speed. A reaction time of 2.0 seconds is used for roads with speeds less than 100 km/h and a reaction time of 1.5 seconds may be used in restricted situations and difficult terrain. NAASRA uses speed prediction procedures to estimate the actual operating speed that is used in determining required stopping sight distances. Australian research concluded that on lower speed facilities, operating speeds are normally higher than the design speeds used in the United States, and at speeds greater than 100 km/h, the two methods produce essentially the same results. Because the stopping sight distance equation is sensitive to changes in design speed, NAASRA introduces a large factor of safety by designing for faster drivers (*41*).

The design of crest vertical curves is controlled by the required stopping sight distance. The minimum K-values (maximum curvature) are found with an eye height of 1.15 meters and an object height of 0.2 meters. Equivalent maneuver times and distances are used to determine minimum stopping sight distances where normal stopping sight distance is difficult or costly to achieve. Australia has a visual capability guideline that states that drivers must be able to recognize a hazard for it to be seen. The literature indicated that an observer can resolve spatial detail to one minute of arc (i.e. the angle subtended by the object height at the eye); however, an angle of five minutes is more typical of the contrast and lighting conditions found on roadways. By translating this requirement into the height of the object which must be visible to be seen, 100 and 200 mm of the object must be above the line of sight at distances of 65 and 130 m, respectively. Thus, small objects will probably not be seen at distances greater than 130 m even with sufficient sight distance. This implies that stopping sight distances for speeds greater than 90 km/h in daylight and 70 km/h at night are beyond the visual capability of most drivers.

Austria. Sight distance policy in Austria is based on an operating speed, known as the project speed, which represents the maximum theoretical speed at a particular location on the road. The maximum project speed corresponds to 100 km/h for two-lane rural roads and ranges from 100 to 140 km/h for multilane roads (*42*). The brake reaction time used in Austria is 2.0 seconds. The following equation is used to represent the braking coefficient at any speed in the deceleration maneuver:

$$f_T(V) = 0.214\ (V/100)^2 - 0.640\ (V/100) + 0.615 \qquad [12]$$

Austria uses the following assumptions in determining aerodynamic drag using Equation (11):

- a drag coefficient (C_w) of 0.46;
- a vehicle frontal area (A) of 2.21 m² for a passenger car, and
- a vehicle mass (m) of 1,175 kg.

Canada. The Canadian stopping sight distance policy is similar to the AASHTO policy but they converted to metric units earlier and the stopping sight distance design values have been rounded differently. Current Canadian practice differs from U.S. practice in that an object height of 0.38 m (representative of a vehicle tail light height) is used to determine curve lengths.

France. The Ministry of Transportation, Division of City Planning and Housing, Division of Roads, and Division of Safety and Road Traffic are responsible for development and promotion of design policies in France (*43*). The standards are mainly for national roads but are generally adapted for city roads by the departmental engineers. Although not documented, the French do not believe that stopping sight distance is important in roadway design, because their accident records indicates that accidents with objects in the roadway are rare events. The most common object struck is a pedestrian, and this type of accident accounts for 5 percent of rural accidents and 8 percent of fatal accidents. These accidents typically occur at night when stopping sight distance is not the factor that limits driver visibility.

Thus, the French design guidelines use an object height of 0.35 m, which represents the taillight height of a vehicle. This height is also sufficient for seeing pedestrians. The French guidelines state that when stopping sight distance is difficult to provide and the roadway has a paved shoulder, an acceptable alternative is to accommodate an evasive maneuver by providing sight distance equal to the lateral displacement for 3.5 seconds at the 85th percentile speed of traffic. This sight distance is measured from the driver's eye height to the pavement surface. For existing roads, they consider the provision of intersection sight distance, visibility of curves, and the lateral displacement rule for stopping sight distance as the most important sight distance concerns.

Germany. Stopping sight distance policy in Germany uses a design speed based on the prevailing 85th percentile speed of traffic (*44, 45, 46, 47, 48*). The brake reaction time used in Germany is 2.0 seconds for rural roads and 1.5 seconds for urban streets. The following equation is used in Germany to represent the braking coefficient at any in the deceleration maneuver:

$$f_T(V) = 0.241\ (V/100)^2 - 0.721\ (V/100) + 0.708 \qquad [13]$$

Germany uses the following assumptions in determining the aerodynamic drag using Equation (11):

- a drag coefficient (C_w) of 0.35;
- a vehicle frontal area (A) of 2.08 m^2 for a passenger car; and
- a vehicle mass (m) of 1,304 kg.

Vertical curve lengths are determined based on an eye height of 1.0 m for passenger cars (or 2.50 m for trucks) and an object height that varies from 0 to 0.45 m as a function of the 85th percentile speed.

Great Britain. The responsibilities for highway design in Great Britain lie with the Department of Transport for national highways and with the County Councils for local roads; however, most County Councils adopt Department of Transport standards and specifications. Stopping sight distance in Great Britain is defined using essentially the same model as given in Equation (1). The roadway design speed is based on geometric constraints and the observed speed of adjoining roadway sections rather than on a general roadway classification.

The perception-reaction time used in Great Britain is 2.0 seconds. The braking distance is based on a coefficient of braking friction intended to avoid excessive discomfort to the driver. A braking coefficient of 0.375 can be achieved in wet conditions on a normally textured surface without loss of control; however, the maximum comfort deceleration rate used in design is based on a braking coefficient of 0.25 (*49*). The driver eye height used to determine vertical curve lengths in Britain range from 1.05 to 2.00 m. The object height is 0.25 m, the height of a rear taillight.

Greece. Sight distance policy in Greece uses a design speed based on the 85th percentile speed of traffic. The brake reaction time used in Greece is 2.0 seconds for rural roads and 1.5 seconds for urban streets. The following equation is used in Greece to represent the braking coefficient at any speed in the deceleration maneuver:

$$f_T(V) = 0.151\ (V/100)^2 - 0.485\ (V/100) + 0.59 \qquad [14]$$

Greece uses the same assumptions as Germany in determining the aerodynamic drag (*50*).

South Africa. Stopping sight distance in South Africa is based on perception-reaction time of 2.5 seconds and an operating speed that is less than the design speed for speeds above 50 km/h. For example, stopping sight distance for a design speed of 120 km/h is based on an assumed operating speed of 101 km/h.

Sweden. The Swedish National Road Administration (SNRA) is responsible for all aspects of the Swedish State Road Network (*51*). Adherence to the design standards for rural roads is required; however, the standards are only recommended for urban highways. The Swedish stopping sight distance model considers design brake-reaction time and design braking friction. The brake-reaction time of 2.0 seconds is the elapsed time from the moment a driver can physically perceive an obstruction on the road until the moment that braking begins between the tires and the road.

Additional variables include a vehicle height of 1.35 m, a driver eye height of 1.1 m, an object height of 0.2 m, and a design visibility angle of one minute of arc. The visibility angle is the minimum optic angle an obstruction must cover to allow the driver of a vehicle to distinguish it in daylight. The required visible portion of the object ranges from 0.01 m of a 0.2 m object at a distance of 50 m to 0.09 m of a 0.2 m object at a distance of 300 m. A headlight height of 0.6 m is used for calculating stopping sight distances at night.

Stopping sight distance generally is not an important parameter for design in Sweden because quantifying the benefits of varying sight distances within their benefit/cost framework is difficult. Through a small-scale study, they determined that accidents increased with an increase in the ratio of the number of locations with less than 300 m sight distance to the total length of the roadway.

Switzerland. Few details regarding Swiss stopping sight distance policy are available; however, it is known that sight distance design is based on a driver eye height of 1.0 m and an object height of 0.15 m (*51*). Sight distance design in Switzerland is based on an operating speed concept similar to the project speed used in Austria.

Comparison of Stopping Sight Distance Design Values. Table A-7 and Figure A-11 compare the minimum required stopping sight distances for the countries whose stopping sight distance policies were reviewed. As shown, the AASHTO design values are near the upper end of the range and the Canadian values are near the lower end of the range. The principal assumptions used in determining stopping sight distance values are the brake reaction time and the braking friction coefficient. All of the countries reviewed use brake reaction time of 2.0 seconds for rural roads, except Australia (for higher speeds only), Canada, South Africa, and AASHTO who use 2.5 seconds.

TABLE A-7. Comparison of Minimum Required Stopping Distancesfor Level Terrain.

Country	t_{pr} sec	Design or Operating Speed (km/h)												
		20	30	40	50	60	70	80	90	100	110	120	130	140
		Stopping Sight Distance (m)												
Australia														
Normal Design	2.5	–	–	–	–	–	–	115	140	170	210	250	300	–
Normal Design	2.0	–	–	–	45	65	85	105	130	–	–	–	–	–
Restricted Design	1.5	–	–	–	40	55	70	–	–	–	–	–	–	–
Austria	2.0	–	–	35	50	70	90	120	–	185	–	275	–	380
Britain	2.0	–	–	–	70	90	120	–	–	215	–	295	–	–
Canada	2.5	–	–	45	65	85	110	140	170	200	220	240	–	–
France	2.0	15	25	35	50	65	85	105	130	160	–	–	–	–
Germany	2.0	–	–	–	–	65	85	110	140	170	210	255	–	–
Greece	2.0	–	–	–	–	65	85	110	140	170	205	245	–	–
South Africa	2.5	–	–	50	65	80	95	115	135	155	180	210	–	–
Sweden	2.0	–	35	–	70	–	165	–	–	–	195	–	–	–
Switzerland	2.0	–	–	35	50	70	95	120	150	195	230	280	–	–
United States	2.5	–	30	44	63	85	111	139	169	205	246	286	–	–

Note: Perception-reaction time (t) values are generally those used for rural roads; for more details see the discussion in the accompanying text.

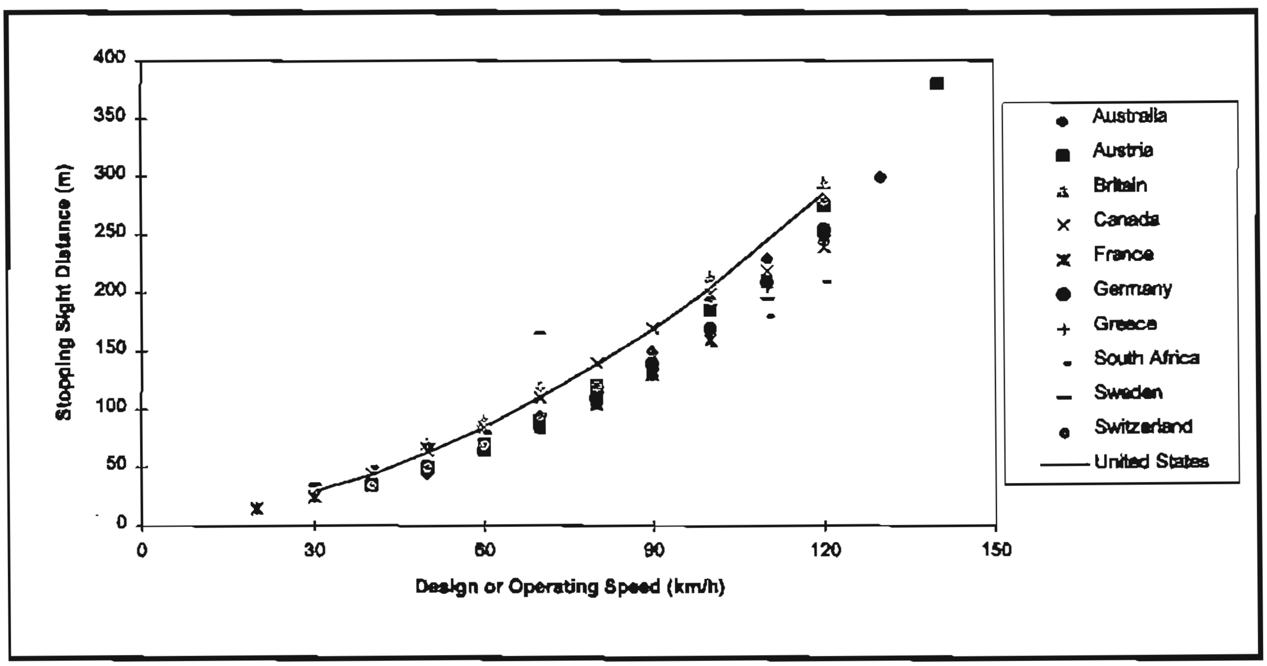


Figure A-11. Comparison of Minimum Required Stopping Sight Distances for Level Terrain.

Table A-8 and Figure A-12 compare the braking coefficients of friction assumed in determining stopping sight distance. In interpreting these data, remember that most of the values in the table and figure represent constant friction during braking, while the Austrian, German, and Greek friction values increase with decreasing speed during the braking maneuver. AASHTO generally has the lowest friction values and the lowest difference in friction values between 50 and 120 km/h.

Table A-9 summarizes the differences between countries in driver eye height and object height for determining required vertical curve lengths. Eye heights for a passenger car driver range from 1.00 to 1.15 m. Object height assumptions are more varied. Australia, Great Britain, Sweden, Switzerland, and the United States each assume a small object with a height in the range from 0.15 to 0.26 m. Canada and France use object heights based on vehicle taillight height ranging from 0.35 to 0.38 m. Germany uses an object height that varies with design speed from zero at low speeds to 0.45 m at high speeds. A unique feature of the Swedish guidelines is that they specify a minimum portion of the object (one minute of arc) that must be visible.

Figure A-13 summarizes the guidelines for determining crest vertical curve lengths from 14 countries. Data for countries besides those directly reviewed in this appendix are based on the work of Krammes and Garnham (*52*). The minimum K-values are based on the required stopping sight distance, as well as eye and object heights. Although many countries specify parabolic vertical curves, most European countries specify circular vertical curves, but for convenience, lay them out in the field as parabolic curves. For a circular vertical curve, the K-value represents the radius of the vertical curve; however, it should be noted that for a given K-value, the alignment of parabolic and circular vertical curves differ by only a few centimeters.

Guidelines for sag curves are compared in Figure A-14. Some countries reviewed, including AASHTO, use sag vertical curve criteria based on headlight height. Other countries consider sag vertical curves as less critical with respect to safety and base sag vertical curve design guidelines on comfort and appearance. As with stopping sight distance, AASHTO crest and sag vertical lengths are near the upper end of the range.

Summary

The alternative stopping sight distance models discussed in this appendix included functional classification models, driver expectancy models, probabilistic models, and international models. **Functional classification models** suggest differences in the operating characteristics experiences for different highway types, and that different stopping sight distance values should be used for different functional classifications to account for these differences. Adoption of a functional classification model would result in the abandonment of the concept that a single stopping sight distance model is appropriate for all highway types and all highway conditions. Different driver, vehicle, and roadway parameters for different classes of roadways would result in a range of design stopping sight distances for a given design speed, rather than one value for all conditions. A concern with this type of model is whether required stopping sight distances would change significantly if and when the highway's functional classification changes.

Using a **driver expectancy model** to determine required stopping sight distances should result in a roadway design that does not create large differentials in driver's operating speeds as they drive along sections of roadway. The basic premise of a driver workload model (which is an indirect measure of driver expectancy) is that by changing the driver's built up expectancy by suddenly increasing their workload, they are more likely to make a mistake (i.e., have an accident). Driver expectancy models determine speed differentials between successive geometric features along the roadway. Such models are useful for checking the consistency of a proposed or existing design and identifying potential problem locations; however, they do not lend themselves to the development of stopping sight distance guidelines.

The **probabilistic models** calculate measures of road safety. For example, the method developed by Navin uses demand as the driver-vehicle system for the highway design parameters, and supply as the parameter values provided by the current standards. Failure occurs when demand exceeds supply. A measure of the margin of safety is the difference between the expected value of the supply and the expected value of the demand. The probability of failure should be evaluated using statistical methods because the safety index equation is not a linear combination. Although intuitively appealing, this model requires frequency distributions for most of the same parameters in the current stopping sight distance model, and would be much more complex to develop and explain. In addition, just because the required stopping sight distance exceeds the available stopping sight distance does not mean that the system fails; i.e., an accident will occur.

Most of the **international models** identified in this study emphasized consistency between design elements. Of particular importance to this research is the fact that several European models also consider operating speed in the determination of the required stopping sight distance. When comparing AASHTO stopping sight distances and curve lengths to those from other countries, AASHTO criteria are near the upper end of the range.

TABLE A-8. Comparison of Longitudinal FrictionCoefficients Used In Stopping Sight Distance Design.

Country	Design or Operating Speed (km/h)									
	30	40	50	60	70	80	90	100	110	120
Australia	–	–	0.52	0.48	0.45	0.43	0.41	0.39	0.37	0.35
Austria	0.44	0.39	0.35	0.31	0.27	0.24	0.21	0.19	0.17	0.16
France	–	0.37	-	0.37	–	0.33	–	0.30	–	0.27
Germany	0.51	0.46	0.41	0.36	0.32	0.29	0.25	0.23	0.21	0.19
Greece	0.46	0.42	0.39	0.35	0.32	0.30	0.28	0.26	0.24	0.23
South Africa										
Sweden	0.46	0.45	0.42	0.40	0.37	0.35	0.33	0.32	0.30	–
Switzerland	–	0.43	0.37	0.33	0.29	0.27	0.25	0.24	0.23	0.22
United States	0.40	0.38	0.35	0.33	0.31	0.30	0.30	0.29	0.28	0.28

Note: The longitudinal friction factors given for Austria, Germany, and Greece are assumed to increase with decreasing speed during the deceleration maneuver. The longitudinal friction factors for other countries represent a constant (i.e., average) over the entire maneuver.

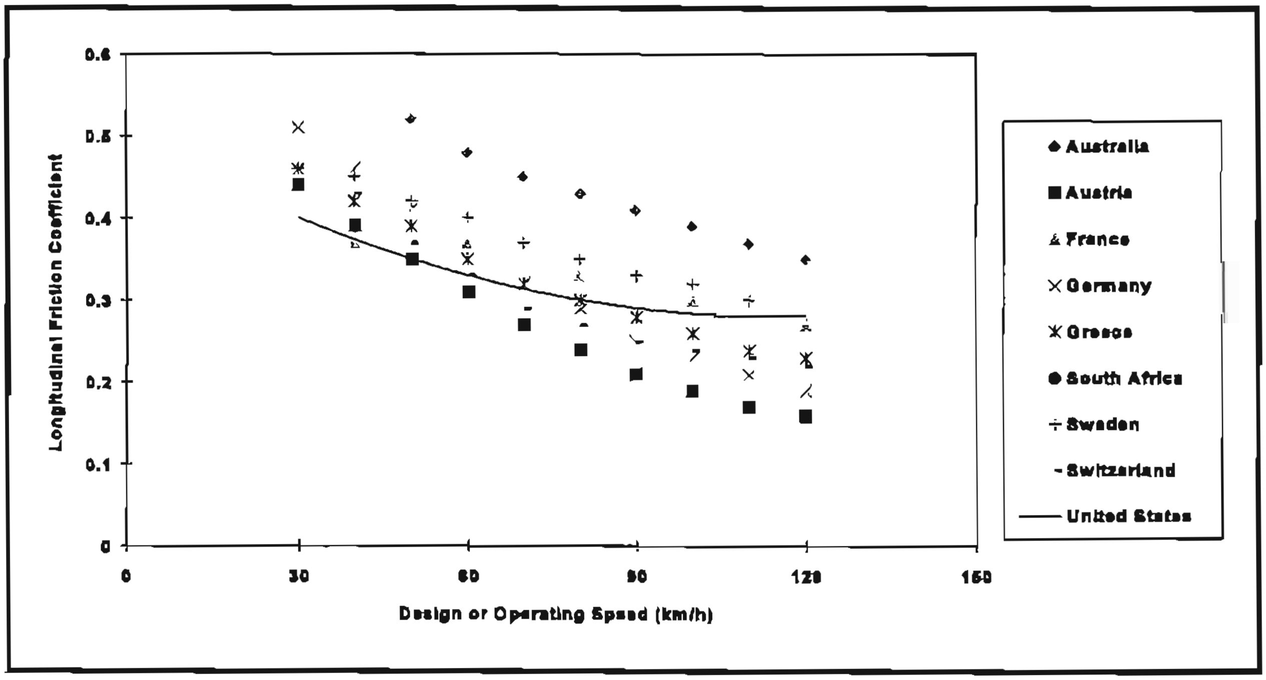


FIGURE A-12. Comparison of Criteria for Longitudinal Friction Coefficient Used in Stopping Sight Distance Design.

TABLE A-9. Comparison of Criteria for Driver Eye Height and Object Height Used in Vertical Curve Design.

Country	Driver Eye Height (m)		Object Height (m)
	Passenger Car	Truck	
Australia	1.15	1.80	0.20
Austria	1.00	–	0.00-0.19
Britain	1.05	–	0.26
Canada	1.05	–	0.38
France	1.00	–	0.35
Germany	1.00	2.5	0.00-0.45
Greece	1.00	–	0.00-0.45
South Africa	1.05	1.80	0.15-0.60
Sweden	1.10	–	0.20
Switzerland	1.10	2.50	0.15
United States	1.07	–	0.15

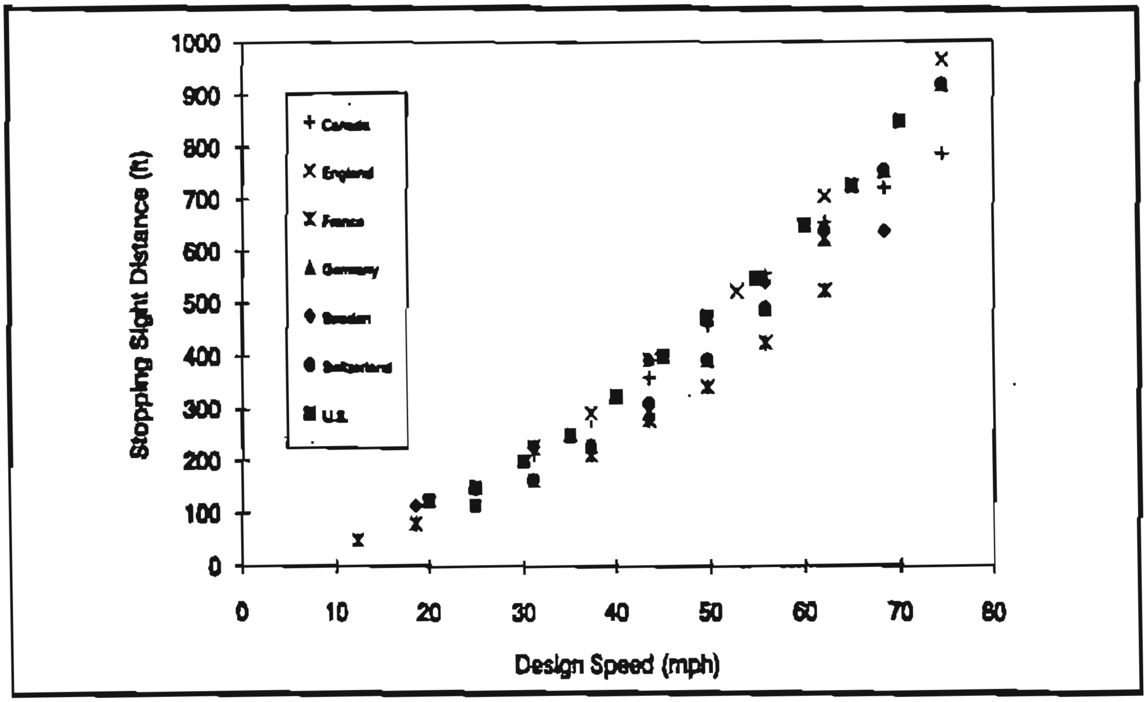


FIGURE A-13. Comparison of Minimum K-Values for Crest Curves

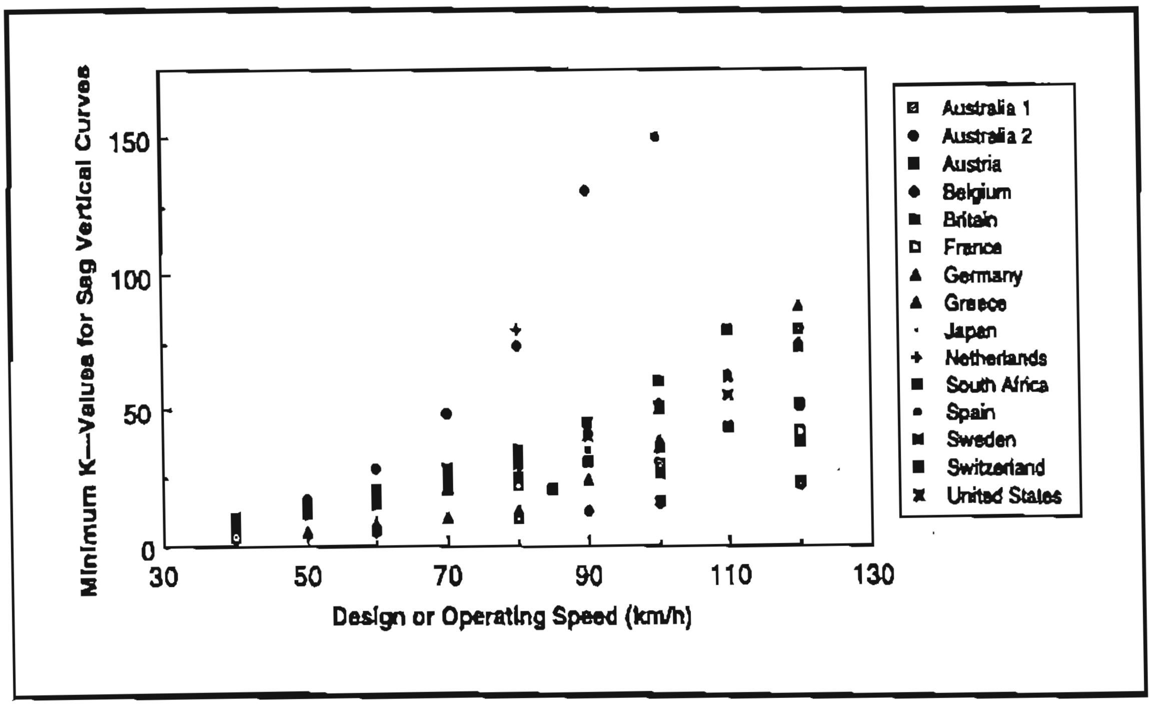


FIGURE A-14. Comparison of Minimum K-Values for Sag Crest Curves

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APPENDIX B -

VEHICLE AND ROADWAY PERFORMANCE

The AASHTO (American Association of State Highway and Transportation Officials) stopping sight distance model (SSD model) uses an equation based on the principles of physics to determine stopping distance. Several factors influence the braking component of the equation. These factors include the vehicle brakes and the pavement surface, both of which are discussed in this appendix. Also included in this appendix are the assumptions in the current AASHTO *Policy on Geometric Design for Streets and Highways* (*1,2*) and an overview of vehicle braking characteristics.

To evaluate the appropriateness of the AASHTO braking distances, a comparison of estimated or measured braking distances to the values currently used for design is needed. This appendix summarizes several recent studies on simulated and actual braking distances of vehicles. The studies are divided into two categories: *Braking Simulation Studies* and *Braking Field Studies*. The *Braking Simulation Studies* include those studies that determined braking distances based on assumptions or extrapolations from research on different components of the braking effort, as well as those studies that determined braking distances from actual tests where a vehicle was braked to a complete stop (the *Braking Field Studies*). Findings from these studies were compared with the AASHTO braking values. This final section of this appendix includes typical friction values and skid numbers obtained from two states and the SHRP database.

AASHTO BRAKING DISTANCE COMPONENT OF STOPPING SIGHT DISTANCE

The braking distance component of the AASHTO stopping sight distance model can be expressed by the following equation:

$$d = \frac{V^2}{30f \pm g} \qquad [1]$$

where:

d = braking distance (ft);
V = initial speed (mph);
f = coefficient of friction between tires and roadways; and
g = percent grade/100.

The friction values selected for use in the current AASHTO stopping sight distance model procedure (see Table B-1) were based on studies conducted by Moyer and Shupe in 1951 (*3*) and were selected to reflect **locked-wheel braking on a poor, wet pavement with worn tires**. The AASHTO (*1*) states that "the friction values used for design should be nearly all-inclusive, rather than average."

TABLE B-1. AASHTO Coefficient of Friction Values (*2*).

Design Speed (mph)	Coefficient of Friction (f)	Braking Distance (feet)
20	0.40	33
25	0.38	55
30	0.35	86
35	0.34	120
40	0.32	167
45	0.31	218
50	0.30	278
55	0.30	336
60	0.29	414
65	0.29	486
70	0.28	583

BRAKING CHARACTERISTICS

Tire-Pavement Friction

Friction is the variable that reflects numerous combinations of vehicle and roadway conditions. It is a reflection of the tire condition, the pavement condition, and the interaction between the tire and the pavement. It varies depending upon the pavement type, whether the pavement is dry or wet or whether the tires are new or worn, and many other conditions. Typical pavement friction values for both dry and wet surfaces are listed in Table B-2.

Water on the pavement surface affects skid resistance. On most paved roadways, water may increase braking distance by one-third or more. This difference increases with higher speeds. In addition to its presence, the amount of water on the pavement surface also affects the coefficient of friction. The two worst conditions occur when the pavement is barely wet or standing under 0.10 inches or more of water. In the first case, a light sprinkle can combine with dirt and grime on the pavement surface to act as a thin film of soap covering the roadway. In the second case, vehicles are susceptible to hydroplaning when reaching speeds in excess of 50 mph. When hydroplaning occurs, the vehicle's tires lose contact with the pavement surface resulting in a drastic reduction in the coefficient of friction, as well as the ability to steer.

A major influence on skid resistance is vehicle speed. On most paved surfaces, the higher the speed, the lower the coefficient of friction. It should be noted that the coefficient of friction is not actually constant throughout the braking maneuver but varies inversely with speed; however, an average or equivalent constant value is used in determining the braking distance.

The coefficient of tire-pavement friction or skid resistance is measured by the amount of drag or retarding force on a vehicle when skidding. It is commonly expressed as either a proportion or a percentage of the vehicle's weight. A heavy vehicle exerts more downward pressure than a light one, and the resultant retarding forces are correspondingly greater. For example, a surface with a coefficient of friction of 0.5 (50 percent) will produce a drag of 1,500 pounds on a vehicle weighing 3,000 pounds and 2,000 pounds on a vehicle weighing 4,000 pounds.

TABLE B-2. Typical Pavement Friction Values (*4*).

Dry Surfaces	Autos	Trucks
Concrete (PC)	0.70–0.90	0.50–0.70
Asphalt (AC)	0.70–0.90	0.45–0.65
Gravel	0.40–0.60	0.40–0.60
Grass or Dirt	0.50–0.70	0.40–0.70
Mud	0.30+	0.30+
Ice (approx. 32 degrees F)	0.06–0.08	0.06–0.08
Ice (less than 32 degrees F)	0.10–0.20	
Wet Surfaces	**Autos**	**Trucks**
Concrete (PC) & Asphalt (AC)	0.45–0.70	0.25–0.55
Polished Surfaces	(0.20)	(0.15)
Gravel	0.30–0.50	0.30–0.50
Grass or Dirt	0.20–0.70	0.30–0.50
Ice (approx. 32 degrees F)	0.06–0.08	0.06–0.08
Ice (less than 32 degrees F)	0.10–0.20	0.10–0.20

Braking and Cornering Friction

Figure B-1 (*5*) illustrates that both braking and cornering friction vary as a function of percent slip. Braking friction is the ratio of the braking force generated at the tire-pavement interface to the vertical load carried by the tire, while cornering friction is the ratio of the cornering force generated at the tire-pavement interface to the vertical load carried by the tire. Percent slip is the percent decrease in angular velocity of a wheel relative to the pavement surface as a vehicle is braking. A freely rolling wheel is operating at 0 percent slip. A locked wheel is operating at 100 percent slip with the tires sliding across the pavement.

Figure B-1 also illustrates that the coefficient of braking friction increases rapidly with percent slip to a peak value that typically occurs between 10 and 15 percent slip. The coefficient of braking friction then decreases as percent slip increases, reaching a level known as the coefficient of sliding friction at 100 percent slip. The coefficient of cornering friction has its maximum value at 0 percent slip and decreases to a minimum at 100 percent slip. Thus, when a braking vehicle's wheels lock, steering capability may be lost due to a lack of cornering friction.

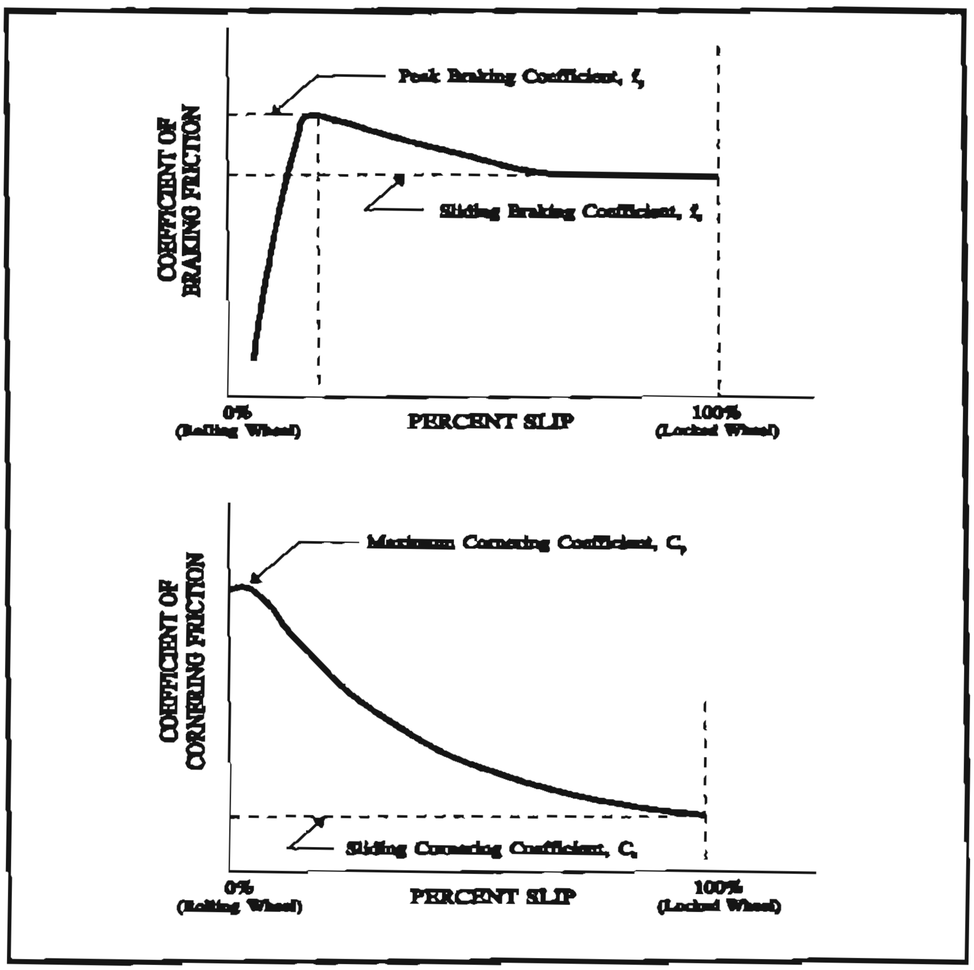

FIGURE B-1. Variation of Braking and Cornering Friction Coefficients With Percent Slip (*5*).

Locked Wheel Versus Controlled Braking

Braking maneuvers can be performed in two general modes: locked-wheel and controlled braking. Another term associated with locked-wheel braking is "panic stop." In a panic stop, the driver "slams-on" the brake pedal and holds it depressed until the vehicle stops. Braking in this mode causes the vehicle to slide or skid over the pavement surface on its non-rotating or "locked" tires. A significant consequence of a panic stop is the loss of control of the vehicle. Locked-wheel braking uses sliding friction, f_s (see Figure B-1), rather than rolling or peak friction. The sliding coefficient of friction takes advantage of most of the friction available from the pavement surface, but is generally less than the peak available friction.

Controlled braking is the application of the brakes in such a way that the wheels continue to roll without locking up while the vehicle is decelerating. Controlled braking distances are governed by the rolling coefficient of friction, which occurs at a percent slip to the left of the peak available friction (see Figure B-1). Drivers generally achieve controlled braking by "modulating" the brake pedal to vary the braking force and avoid locking the wheels. Harwood et al. (*5*) noted that due to the steep slope of the braking friction curve to the left of the peak, available friction and the braking techniques used by drivers to avoid wheel lock up, the average rolling friction coefficient is generally less than the sliding friction coefficient. Thus, driver controlled braking distances are usually longer than locked-wheel braking distances, although theoretically they would be less if drivers were able to use peak braking friction.

The difference in braking distance using the two different approaches can be significant. Figure B-2 illustrates the differences that occurred using locked-wheel (panic) and controlled (modulated) braking for a long wheelbase, three-axle bobtail tractor (*6*). Note the almost 50 percent increase in braking distance under controlled braking.

BRAKE TYPES AND USAGE

According to a recent Federal Highway Administration (FHWA) report (*5*), air brakes are the most common braking systems. Air brakes use compressed air to transmit and amplify the driver's input from the brake pedal to the brakes on individual wheels. The use of air as an amplifying medium results in a slight delay in the system response due to the compressibility of air. Once the brake pedal is released, the air in the system is expelled to the atmosphere and replaced by air from a compressor on board the truck.

In contrast to air brakes, hydraulic brakes provide an almost immediate response to a driver's brake pedal action. Hydraulic brakes can be "pumped" (as opposed to air brakes) during a controlled stop. According to Kirkbride and Radlinski (*7*), hydraulic brake systems are used on most single unit vehicles with Gross Vehicle Weight Ratings (GVWR) of 26,000 pounds or less and on many single unit vehicles with GVWR between 26,000 and 33,000 pounds.

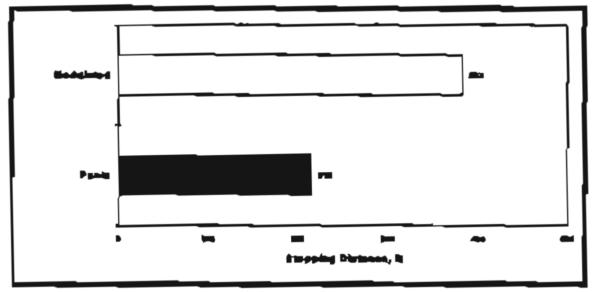

FIGURE B-2. Stopping Distance From 60 Miles Per Hour on Dry Pavement For a Long Wheelbase Three-Axle Bobtail Tractor (*6*).

Hydraulic brakes are used to a lesser degree above GVWR 33,000 pounds, but are available on trucks with GVWRs up to 46,000 pounds.

Antilock braking systems (ABS) were developed in the 1950s by the aircraft industry for use during wet weather landing maneuvers. By the 1970s, motor vehicle manufacturers began developing their own systems for use in passenger cars and light trucks. Technological advances, especially with the microprocessor, have resulted in the widespread use of ABS in a wide range of new vehicle models. Ford, Chrysler, and General Motors have expended considerable effort and funds in the research and development of a cost-effective ABS for their vehicles. All three manufacturers have experienced increased production and sales of ABS-equipped vehicles due in part to their efforts to improve antilock brake components.

Recent advancements in computer electronics have improved the compactness of the ABS resulting in a more competitive and marketable product. Antilock braking systems are designed to automatically modulate brake pressure to prevent over-rapid deceleration of the vehicle's wheels. With conventional braking systems, the wheels may lock on the pavement and result in vehicle instability (if the wheel or wheels are allowed to decelerate too rapidly during hard braking maneuvers). Antilock braking systems prevent wheels from locking by using wheel sensors that determine wheel speed and slip. Electronic controllers then maintain the braking forces at an optimum tire-to-road adhesion by modulating the brake pressure on the wheels. The result is shorter braking distances and increased vehicle control during braking.

Figure B-3 shows the percentage of new vehicle sales with antilock brakes. Between 1987 and 1990, passenger cars equipped with ABS accounted for 3.6 to 7.6 percent of the almost 6.9 million passenger car sales each year. Light trucks showed a significant increase in antilock brake installation, from 19.9 to 79.4 percent of the approximately 4.2 million light truck sales each year; however, it should be noted that Figure B-3 represents new vehicle sales and the percentage of ABS vehicles in the current fleet is much smaller.

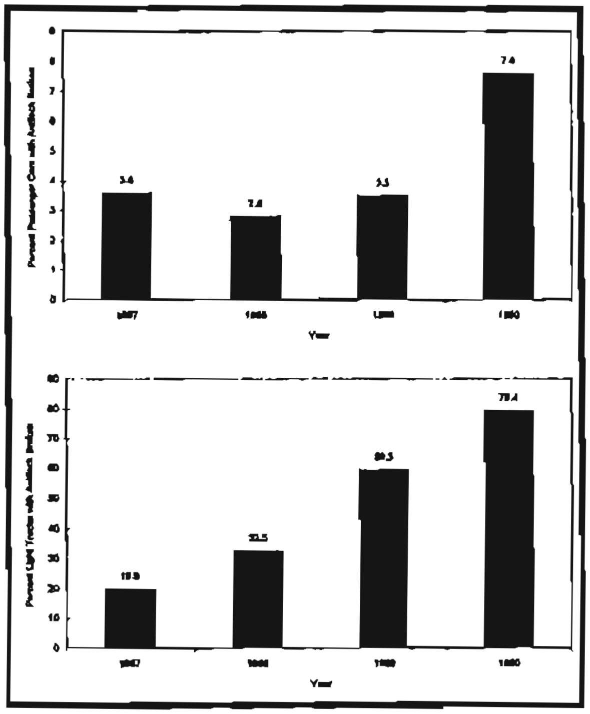

FIGURE B-3. Antilock Brakes in Passenger Cars and Light Trucks (New Sales).

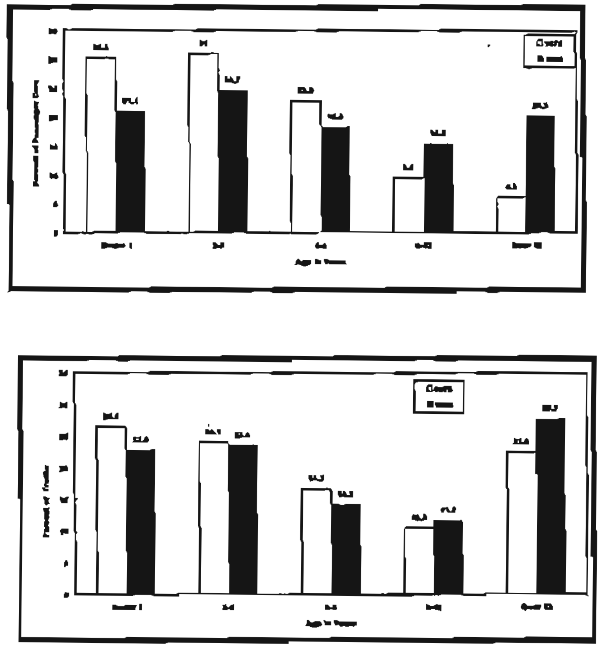

FIGURE B-4. Age of Passenger Cars and Truck Fleets.

The vehicle fleet in the United States is continually getting older. In 1990, the average age of automobiles on the road was 7.8, whereas the average age in 1970 was 5.6 years. Figure B-4 shows the percentage of passenger cars by vehicle age in 1970 and 1990. Note that the number of vehicles older than 12 years has more than doubled between 1970 and 1990. The percentage of trucks per age group from 1970 and 1990 is also shown in Figure B-4. The average age of trucks in 1970 and 1990 is 7.3 and 8.0, respectively. Using this information, it appears that it will take 10 to 15 years to replace 80 percent of the vehicle fleet with new vehicles.

The magnitude of the ABS use in the future is heavily dependent upon U.S. government-related activities. On September 28, 1993, the National Highway Traffic Safety Administration (NHTSA) published a Notice of Proposed Rulemaking, proposing to require ABS on all new commercial vehicles. The final rule was published by NHTSA on March 10, 1995, and requires ABS on all newly manufactured commercial vehicles as follows: (1) on/after March 1, 1997, truck tractors; (2) on/after March 1, 1998, all other air braked vehicles; and (3) on/after March 1, 1999, all hydrauli cally braked vehicles. Also, on March 10, 1995, the F-HWA's Office of Motor Carriers published a notice of intent concerning maintenance policies for antilock brakes.

BRAKING MAINTENANCE

On April 29, 1992, the National Transportation Safety Board adopted the Heavy Vehicle Airbrake Performance Safety Study (*8*). The study focused on brake system issues and made recommendations that address the systemic problems associated with heavy vehicle brake-related accidents. The accident and inspection data highlighted three safety issues: the difficulty of keeping commercial vehicle brake systems adjusted; the problem of maintenance deficiencies; and the role of brake system components in vehicle instability accidents. As a result of the study, recommendations were issued to several organizations, including the NHTSA, the American Trucking Associations, and others.

Brake Adjustment. Poor adjustment of air brakes has been found to substantially increase braking distance. NHTSA investigated the effects of air brake adjustment on truck braking performance in the early 1980s (*9, 10*). Trucks were tested with all of their brakes adjusted for optimum performance and with their brakes adjusted to be just within allowable limits. The results (see Table B-3) indicated that braking distances can increase by more than 30 percent due to brake maladjustment.

NHTSA also reported on surveys conducted in California and Maryland to determine the percentage of trucks with brake(s) out of adjustment. The California studies were conducted in 1977 and 1981 and the Maryland study was conducted in 1981. California's 1981 study was a follow up survey conducted after an increase in enforcement efforts. Table B-4 presents the findings from the surveys. Note the large percentage of vehicles with one or more brakes out of adjustment.

Brake Lining Temperature. The NHTSA study (*9, 10*) also determined the effect of brake lining temperature on braking distance. Table B-5 lists the results at different temperatures for the three-axle single-unit truck for both fully adjusted brakes and for brakes readjusted to the limit of the specifications. Fully adjusted brakes were found to be less sensitive to temperature than maladjusted brakes.

Disconnection of Front-Axle Brakes. Although illegal, truck drivers have been known to disconnect front-axle brakes because of the possibility of losing control if the front-axle brakes lock in an emergency stopping situation. Tests by NHTSA have shown that trucks with disconnected front brakes require 20 to 25 percent greater braking distance (*11*).

Automatic Limiting Valves for Front-Axle Brakes. A component that has been added to braking systems in recent years is an automatic limiting valve (ALV) for the front-axle brakes. According to NHTSA (*12*), approximately two-thirds of post-1980 combination unit trucks are equipped with automatic limiting valves. The advantage of an automatic limiting valve is that it reduces the possibility of wheel lock on the steering axle, which means that the driver retains steering control during emergency application of the brakes. The main disadvantage is that an automatic limiting valve reduces the braking capability of the truck which lengthens the braking distance. Table B-6 presents data for controlled stops with and without automatic limiting valves.

TABLE B-3. Effect of Brake Adjustment on Braking Distance.

Vehicle	Weight (pounds)	Speed (mph)	Brake temp (°F)	Average braking distance (ft)		Percent Increase
				Fully Adjusted	Adjusted to Limit	
Single-unit truck with two axles	27,500	55	< 200	219	283	29
Tractor-semitrailer (3S2)	80,500	60	< 200	256	319	25
Single-unit truck with three axles	55,000	60	< 200	342	458	34

TABLE B-4. Surveys of Brake Adjustment Status for In-use Vehicles.

Date	Survey Location	Number of Vehicles	Vehicles with one or more brakes out of adjustment	Vehicles with 40% or more brakes out of adjustment
1977	California	1433	68%	19%
1981	California			
	w/o auto slacks	94	47%	15%
	w/ auto slacks	96	42%	9%
1981	Maryland	80	69%	28%

TABLE B-5. Effect of Brake Lining Temperature on Braking Distance.

Single-unit, 3-axle truck 55,000 pounds, 60 mph	Braking distance for brake lining temperatures				Increase between 150 and 400°F
	150°F	200°F	300°F*	400°F	
Fully adjusted brakes	342	351	366	393	15%
Adjusted to limit	458	519	625	692	51%

* *Brake lining temperatures as high as 400°F are not unusual in normal operation and can increase considerably in city or mountain driving.*

TABLE B-6. Braking Distances for Trucks With and Without an Automatic Limiting Valve (*12*).

Characteristics of test	Vehicle type	Braking Distance (ft)	
		With ALV	Without ALV
60 mph, empty, straight-line stop	Single-unit, three-axle truck	440	355
	Bobtail tractor with three axles	418	324
50 mph, empty, 500 feet radius curve, wet asphalt	Single-unit, three axle truck	268	233
	Tractor-semitrailer (2S1)	260	224
	Bobtail tractor with two axles	308	249
	Auto transport truck	251	181
18 mph, loaded 500 ft ice radius curve	Tractor-semitrailer (3S2)	273	253
	Tractor-semitrailer (2S1)	213	179

RECENT BRAKING SIMULATION STUDIES

NCHRP 270, Olson et al. (1984)

A key argument in NCHRP 270 is that drivers traveling at high speed on a wet pavement, to be inclined to brake sufficiently so as to lock the wheels on their vehicles and hold them at lockup (*13*). Rather, they will probably modulate their brakes so as to retain an ability to steer the vehicle. This argument formed the basis of their recommended braking distances, which were significantly longer than the current AASHTO values.

Based on an "examination of the influences of pavement, tire, vehicle, and driver properties on vehicle braking distance," Olson et al. (*13*) developed sets of equations that can be used to predict braking capabilities of cars and trucks operating on poor, wet roads. These equations were used in "connection with a numerical integration algorithm" to determine braking distances. Integration is needed because the frictional characteristics at the tire-road interface change as the vehicle's velocity changes during a stop. Aerodynamic drag also changes as velocity changes.

An approximation can be used to eliminate the need for numerical integration. Table B-7 lists the equations that reproduce the values presented in the several references based on the NCHRP 270 work (*13,14,15*). The equations are grouped by the following areas: roadway characteristics, tire properties, vehicle properties, and driver characteristics. Tables B-8 and B-9 show sample calculations for passenger cars and trucks, respectively. These calculations were used to verify the values in NCHRP 270. Following is a summary of the NCHRP 270 discussion describing these equations.

Roadway Characteristics. The available pavement friction is characterized by ASTM skid numbers. NCHRP 270 included equations (see Equations 4 and 5 in Table B-7) that express the relationship between skid numbers for various speeds and quality of pavement. These equations were from other research studies (*16,17*). The mean pavement texture depth (MD) and the skid number (SN) represent the quality of the pavement. A texture depth of 0.015 inches and a skid number of 28 were selected to represent a 15th percentile, wet road in the example calculations.

Tire Properties. Based on other research, Olson et al. stated that new car tires are approximately 1.2 times more effective than the ASTM tire (used to determine skid numbers) and that truck tires are approximately 0.7 times as effective as car tires. These values were used to determine sliding friction coefficients in Equations 8 and 9. The relationship between maximum rolling friction (f_p) and sliding friction (f_s) are shown in Equations 11 and 12. The differences between the frictional capability of worn tires and new tires is accounted for in Equation 13. Truck tires are assumed to have the same wear characteristics as those attributed to car tires when tire groove depth wears below 12/32 inches.

Vehicle Properties. Braking efficiency (BE) represents "the influences of the proportioning of the braking effort amongst the various wheel locations and the overall distribution of mass throughout the vehicle." The braking systems used in new passenger cars are designed to be very efficient in utilizing the peak friction available at the tire-road interface. Based on data from Radlinski and Flick (*18*), the average efficiency of 1982 passenger cars was found to be approximately 91 percent. Equation 15 is used to determine braking efficiency of trucks. NCHRP 270 reported that the braking efficiencies of empty heavy trucks will range from 0.55 to 0.59 as peak friction varies from 0.43 to 0.21.

Driver Characteristics. Driver control efficiency (CE) predicts the ability of the driver to utilize the deceleration capability afforded by friction and braking efficiency. Equation 19 was developed based on experiments conducted by Mortimer et al. (*19*). Automobile drivers were asked to stop as quickly as possible while following a slightly curving 10-ft lane. The results showed that drivers could not utilize the maximum braking capabilities of their vehicles because they could not modulate their brakes well enough to use the available tire-road friction and simultaneously avoid loss of directional control due to wheel lockup.

Due to the lack of information on braking control efficiencies of truck drivers, NCHRP 270 performed a limited set of experiments. The results of these studies indicated that professional drivers of heavy trucks could usually achieve 62 percent of the braking capabilities of their vehicles in an unladen condition during a braking-in-a-turn maneuver. This group of drivers failed to stay within a 12-ft lane in approximately one-sixth of their attempts to stop from 40 mph.

Predicted Braking Performance. Figures B-5 and B-6 illustrate the braking performance of passenger cars and trucks, respectively. Figure B-5 shows values for type of braking (locked-wheel or controlled), type of tire condition (new tire or worn), type of driver control, and the braking distances given in 1990 AASHTO policy (*2*). Figure B-6 shows truck braking distances for type of braking and type of tire condition.

Heavy Truck Braking, Harwood et al. (1990)

Stopping sight distances must take into account the fact that trucks cannot make a locked-wheel stop without the risk of losing steering control. The process of bringing a truck to a stop requires a complex interaction between the driver, brake system, truck tires, dimensions and loading characteristics of the truck, and the pavement surface characteristics. Harwood et al. (*5*) presented a detailed discussion on this complex interaction. Several of their assumptions were based on the NCHRP 270 work. Following is a summary of their discussion.

Pavement and Tire Properties. The shape of the braking friction curve in Figure B-1 is a function of both pavement and tire properties. Highway agencies generally measure pavement friction by means of locked-wheel skid tests with a standard tire. Olson et al. (*13*) estimated the peak coefficient of friction for truck tires from the sliding coefficient of friction found in locked-wheel skid tests. Because truck tires are designed primarily for wear resistance, they tend to have lower wet friction coefficients than passenger car tires. Olson et al. (*13*) estimated that truck tires have coefficients of friction that are about 70 percent of those of passenger car tires; however, passenger car tires generally have coefficients of friction that are about 120 percent higher than the standard tires used in skid testing. The coefficient of friction for truck tires decreases as the tires wear and their tread depth decreases. New truck tires have tread depths of 15/32 inches for ribbed tires and 31/32 inches for lug type tires. Based on a report by Dijks (*20*), Olson et al. (*13*) assumed, that the truck tire tread wear has very little effect on their frictional properties until the tread depth falls below 12/32 inches. Tire tread depth has little effect on friction values for pavements with high macro texture, but friction values decrease substantially with tread depth for smooth, poorly textured pavements.

Braking Efficiency. Current truck braking systems are limited in their ability to take advantage of the available friction at the tire-pavement interface. Fancher (*14*) estimated that the braking efficiency for single-unit trucks is between 55 and 59 percent of the available friction. Braking efficiency is influenced by disconnected front brakes (*21*), automatic limiting valves which limits the amount of braking achievable on the front axle (*12*), and antilock braking systems. Enforcement activities to assure that front brakes are not disconnected have been increased.

Driver Control Efficiency. Most truck drivers have little or no practice in emergency braking situations. This lack of expertise in modulating their brakes results in braking distances that are longer than the vehicle's capability. Olson et al. (*13*) evaluated the effect of driver efficiency on braking distance using both experienced test drivers and professional truck drivers without test track experience. The study found that the driver efficiencies ranged from 62 to 100 percent of the vehicle capability. The braking performance of the drivers tended to improve during testing as the drivers gained experience in modulating the brakes. Antilock brake systems could eliminate the driver efficiency concern by providing computer-controlled modulation of the brakes to achieve minimum braking distance.

TABLE B-7. Equations to Calculate Braking Distances in NCHRP 270.

$$Braking\ Distance = V_i^2 / 30f \qquad [2]$$

$$f = f_r * CE \qquad [3]$$

where terms are as described below:

ROADWAY CHARACTERISTICS

SN_{V_a}	=	$SN_{40}e^{P(V_a-40)}$	[4]
P	=	$-0.0016(MD)^{-0.47}$	[5]
Va	=	$0.707V_i$	[6]

where:

SN_{V_a} = ASTM skid number at V_a mph
SN_{40} = ASTM skid number at 40 mph
P = Normalized skid number gradient (see Equation 7)
V_a = approximation of velocity that eliminates the need for numerical integration
V_i = initial velocity (mph)
MD = mean pavement texture depth (inches) measured by the sandpatch method

TIRE PROPERTIES

f_s	=	$1.2\times SN_{V_a}/100$, for car tires	[8]
f_s	=	$1.2\times 0.7\times SN_{V_a}/100$, for truck tires	[9]
f_p	=	$0.2 + 1.12f_s$, for car tires	[10]
f_p	=	$1.45f_s$, for truck tires	[11]
TD	=	$1+[(5.08MD-0.008045Va)\times(1-{}^{x/32}/_{12/32})]$	[12]
fx	=	$f_p\times TD$	[13]

where:

f_s = sliding friction coefficient (tire to road)
f_p = peak (maximum) rolling friction coefficient (tire to road)
TD = tire tread depth adjustment factor
x = tire tread depth in 32nds of an inch (for tires with greater than 12/32 inch tread depth, use x=12)
f_x = tire-to-road friction with x/32 inches of tread groove depth
SN_{40}, MD, and V_a are explained above

TABLE B-7. (continued) Equations to Calculate Braking Distances Based on Information in NCHRP 270.

VEHICLE PROPERTIES

BE = 0.91 for cars [14]
BE = $0.47/(0.75+0.23f_p)$ for trucks [15]
$f_a = 0.5C_aV_i^2$ for cars [16]
$f_a = 0.00238 \times A_f \times C_D \times Va^2/W$ for trucks [17]
$f_r = f_x \times BE + f_a$ [18]

where:
BE = vehicle braking efficiency
f_a = deceleration due to aerodynamic drag
$C_a = 10^{-5}$ for passenger cars
A_f = frontal area
C_D = drag coefficient
W = weight
f_r = friction based on tire and vehicle properties
f_p, V_i, V_a, and f_x are explained above

DRIVER CHARACTERISTICS

CE = $0.444f_{x@Vi} \times BE + 0.267$ for passenger cars [19]
CE = 0.62 for trucks [20]

where:
CE = driver braking control efficiency
$f_{x@Vi}$ = uses Equations 4 to 13 except V_i (initial velocity) values are used rather than V_a (approximate velocity) values in each relevant equation (for example, V_i would be used in Equation 4 to calculate the skid number used in Equations 8 or 9).

FOR LOCKED-WHEEL BRAKING

Similar to above with the following exceptions:

BE and CE = 1.0

for tire properties, $f_{x\,locked} = f_x \times TD$ [21]

TABLE B-8. Sample Calculations for a Passenger Car Using Equations in Table B-7.

Passenger Car, Controlled Stopping, 2/32 tires, 60 mph

Assumed/Given:

V_i = 60 mph
SN_{40} = 28
MD = 0.015
x = 2
C_2 = 10^{-5} for passenger cars

Calculations:

$P = -0.0016(0.015)^{-0.47} = -0.0115$
$V_a = 0.707(60) = 42.42$
$SN_{V_a} = 28e^{-0.0115(42.42-40)} = 27.23$
$f_s = 1.2\times 27.23/100 = 0.33$
$f_p = 0.2 + 1.12(0.33) = 0.57$
$TD = 1+[\{((5.08)(0.015)-(0.008045)(42.42)\}\times\{(1-({}^{2/32}/_{12/32}\}] = 0.84$
$f_x = 0.57\times 0.84 = 0.48$
BE = 0.91 for cars
$f_a = 0.5(10^{-5})(60)^2 = 0.018$
$f_r = 0.48\times 0.91 + 0.018 = 0.45$

$SN_{V_i} = 28e^{-0.0115(60-40)} = 22.24$
$f_{s@Vi} = 1.2\times 22.24/100 = 0.27$
$f_{p@Vi} = 0.2 + 1.12(0.27) = 0.50$
$TD_{@Vi} = 1+[\{((5.08)(0.015)-(0.008045)(60)\}\times\{(1-({}^{2/32}/_{12/32})\}] = 0.76$
$f_{x@Vi} = 0.50\times 0.76 = 0.38$
$CE = 0.444(0.38)(0.91) + 0.267 = 0.42$

$f = 0.45\times 0.42 = 0.189$

BD = $60^2/(30\times 0.189)$ = 635 feet <——————

Passenger Car, Locked-Wheel Stopping, 2/32 tires, 60 mph

Use above information with the following changes:

$f_{x,locked} = 0.33\times 0.84 = 0.28$
$f_r = 0.28\times 1.00 + 0.018 = 0.30$
$f = 0.30\times 1.00 = 0.30$
BD = $60^2/(30\times 0.30)$ = 400 feet <——————

TABLE B-9. Sample Calculation for a Truck Using Equations in Table B-7.

Truck, Controlled Stopping, 2/32 tires, 60 mph

Assumed/Given:

V_i = 60 mph
A_f = 100 ft^2
SN_{40} = 28 C_D = 0.8
MD = 0.015 x = 2
W = 14,600 lb for an empty truck

Calculations:

$P = -0.0016(0.015)^{-0.47} = -0.0115$
$V_s = 0.707(60) = 42.42$
$SN_{V_s} = 28e^{-0.0115(42.42-40)} = 27.23$
$f_s = 1.2 \times 0.7 \times 27.23/100 = 0.23$
$f_p = 1.45(0.23) = 0.33$
$TD = 1+[\{(5.08)(0.015)-(0.008045)(42.42)\} \times \{(1-(^{2/32}/_{12/32}))\}] = 0.84$
$f_x = 0.33 \times 0.84 = 0.28$
$BE = 0.47/(0.75+0.23 \times 0.33) = 0.58$
$f_a = 0.00238(100)(0.8)(42.42^2)/(14,600) = 0.02$
$f_r = 0.28 \times 0.58 + 0.02 = 0.18$
$CE = 0.62$
$f = 0.18 \times 0.62 = 0.11$
BD $= 60^2/(30 \times 0.11) = $ **1091 feet** ←

Truck, Locked-Wheel Stopping, 2/32 tires, 60 mph

Use above information with the following changes:

$f_{x,locked} = 0.23 \times 0.84 = 0.19$
$f_r = 0.19 \times 1.00 + 0.02 = 0.21$
$f = 0.21 \times 1.00 = 0.21$
BD $= 60^2/(30 \times 0.21) = $ **571 feet** ←

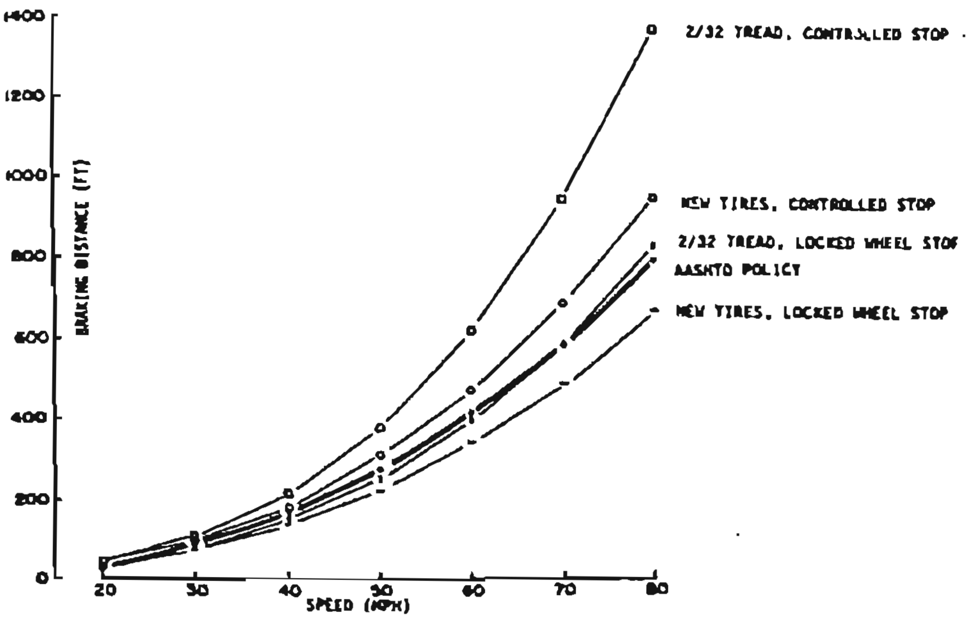

FIGURE B-5. Predicted Passenger Car Braking Distances on a Poor, Wet Road (*13*).

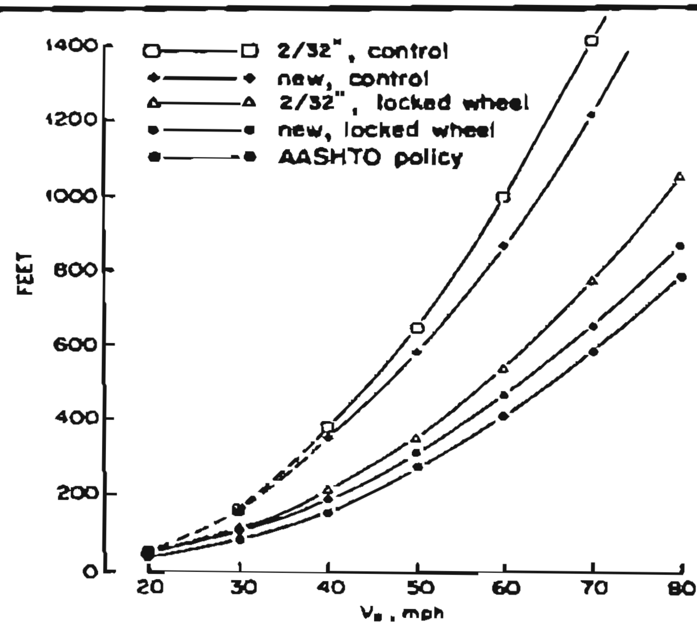

FIGURE B-6. Predicted Truck Braking Distances on a Poor, Wet Road (*14*).

Truck Braking Distances. Harwood et al. (*5*) evaluated three different scenarios for determining truck braking distances—an empty tractor-trailer truck with a conventional braking system and an inexperienced driver; the same truck and braking system operated by an experienced driver; and finally, the same truck with an antilock braking system. The antilock stopping distances were based on braking tests conducted by NHTSA for the FHWA study. The results for the three scenarios, shown in Table B-10, indicate that braking distances for trucks with antilock brakes are similar to the AASHTO criteria for passenger cars.

Harwood's report included a list of candidate stopping sight distance criteria. The values were based on a compromise between best- and worst-driver performance and the assumption that antilock brake systems do not come into general use. A cost-effectiveness concluded that stopping sight distance criteria for trucks may only be applicable under limited conditions in new construction or major reconstruction projects on highways with relatively high truck volumes.

Pavement-Truck Tire Interaction, Kulakowski et al. (1991).

A 1992 FHWA report documented the results from tests on six common truck and bus tires and computer simulation of truck braking (*22*). It also included discussions on how the new findings affect the recommendations from the 1990 FHWA study by Harwood et al. (*5*). Regression analysis and the results from the computer simulation were used to determine the controlled braking distance from 55 mph for any combination of variables and interactions studied. Table B-11 lists the adjustments that should be added or subtracted from the truck mean braking distance of 381.81 feet. For example, the controlled braking distance from 55 mph to a stop for a truck with a walking beam suspension and bias rib tires on a tangent alignment with rough pavement under wet conditions would be calculated as follows:

$$\text{Braking distance} = 381.81 - 4.5 + 6.24 - 7.70 + 9.72 + 4.10 - 4.11 + 1.10 + 1.76 - 2.29 - 3.15 + 1.39 = \mathbf{384.37\ feet}$$

The authors commented that each of the main effects listed in Table B-11 is in a direction that would be expected from existing knowledge of truck braking distances.

The study's findings documented that there are substantial differences in braking distances between passenger cars and trucks. They stated that only a portion of this difference is due to tire design, very little attributable to pavement roughness, and none is attributable to the friction characteristics of the pavement itself. The authors concluded that most of the difference in braking distances is due to differences in vehicle design and braking system design.

The authors cautioned that the lack of real-world data makes it difficult to quantify driver braking performance. They concluded that the results of the truck braking distance obtained from the computer simulation model do not provide sufficient basis for modifying the conclusions and recommendations of the previous FHWA-sponsored truck braking study (*5*). These conclusions included the following:

- Current AASHTO stopping sight distance criteria are adequate for trucks with antilock braking systems.
- Current AASHTO criteria are not adequate for trucks with conventional braking systems and poor performance drivers. Depending on the initial speed, a driver with 70 percent control efficiency (a poor but not extreme value) requires 25 to 425 feet of additional stopping sight distance.
- Revised stopping sight distance criteria for trucks with conventional braking systems would be cost effective only for new construction or major reconstruction projects on rural, two-lane roadways that carry more than 800 trucks/day and rural freeways that carry more than 4,000 trucks/day.

RECENT STUDIES ON BRAKING FIELD TESTS

Antilock Braking, Radlinski and Bell (1986)

Tests were conducted by NHTSA to evaluate the braking performance of a two-axle truck with and without an antilock braking system (*21*). Straight line stops, stops in a turn, and stops in a lane change were studied. Table B-12 lists the results for the different types of brakes, surfaces, loadings, and stopping maneuvers. In general, the authors concluded that the antilock braking system provided improved stopping capability. Most of the stops with antilock brakes were performed in a shorter distance than without the antilock brakes. The ABS-equipped vehicle was also under the control of the driver. For a straight-line stop from 60 mph on a wet polished concrete pavement (SN_{40} is approximately 30), there was a 15 percent reduction in braking distance between controlled braking and antilock braking.

Heavy Vehicle Braking, Radlinski (1987)

Radlinski in 1987 published a paper that provided a synthesis of available information on the braking performance of heavy (on-highway) vehicles in the U.S. (*6*). The paper contrasted the braking performance of heavy vehicles with that of passenger cars and was divided into four major areas—braking systems, heavy vehicle drivers, braking performance, and maintenance practices. A summary of his findings are presented in the following sections.

Braking System Design/Performance. The design of a heavy vehicle's braking system is a complex task. It is also more difficult to optimize braking performance for heavy vehicles than it is for passenger cars. Large weight distribution changes that occur in heavy vehicles make it

TABLE B-10. Braking Distances for Trucks (4).

Design Speed (mph)	AASHTO Criteria for Passenger Cars (feet)	Braking Distances for Trucks (feet)		
		Inexperienced Driver	Experienced Driver	Antilock Braking System
20	33	77	48	37
30	86	186	115	88
40	167	344	213	172
50	278	538	333	267
60	414	744	462	375
70	583	1013	628	510

TABLE B-11. Predicted Truck Braking Distance at 55 mph (22).

Braking Distance (overall mean) = 381.81 feet			
Main Effects			
Suspension		**Pavement Surface Condition**	
Four spring	4.50	Dry	-4.10
Walking beam	-4.50	Wet	4.10
Tire Type		**Alignment**	
Bias rib	6.24	Nontangent	7.70
Radial rib	-2.04	Tangent	-7.70
Low profile	-4.20		
		Roughness	
		High	9.72
		Low	-9.72
Two-Way Interactions			
Suspension/Alignment Interaction		**Alignment/Pavement Surface Condition Interaction**	
Four spring/nontangent	-4.11	Nontangent/dry	-2.29
Four spring/tangent	4.11	Nontangent/wet	2.29
Walking beam/nontangent	4.11	Tangent/dry	2.29
Walking beam/tangent	-4.11	Tangent/wet	-2.29
Suspension/Pavement Surface Condition Interaction		**Alignment/Tire Type Interaction**	
Four spring/dry	1.10	Nontangent/bias rib	3.15
Four spring/wet	-1.10	Nontangent/radial rib	-0.66
Walking beam/dry	-1.10	Nontangent/low profile	-2.49
Walking beam/wet	1.10	Tangent/bias rib	-3.15
		Tangent/radial rib	0.66
Alignment/Roughness Interaction		Tangent/low profile	2.49
Nontangent/high	-1.76		
Nontangent/low	1.76	Roughness/Pavement Surface Condition Interaction	
Tangent/high	1.76	High/dry	-1.39
Tangent/low	-1.76	High/wet	1.39
		Low/dry	1.30
		Low/wet	-1.39

Note: Truck braking distance for a controlled stop from 55 mph can be computed by adding the effects shown for the appropriate combination of factors and interactions to the overall mean of 381.81 feet.

TABLE B-12. Braking Distances for a Two-Axle Truck With and Without ABS (*21*).

Surface	Skid Number	Maneuver	Speed (mph)	Braking Distance (ft)					
				Loaded			Empty		
				AL*	LW	CB	AL	LW	CB
Dry Asphalt	80	St. Line	60	297	318	302	195	179	205
Dry Asphalt	80	St. Line	35	90	94	92	64	64	72
Wet Asphalt	60	St. Line	60	278	315	338	226	248	222
Wet Asphalt	60	St. Line	35	71	66	78	69	71	85
Wet Pol Con.	30	St. Line	60	333	—	405	325	373	356
Wet Pol Con.	30	St. Line	35	87	97	117	93	98	100
Wet Jennite	20	St. Line	35	134	216	232	148	232	155
Split Mu	20/60	St. Line	35	110	107	129	110	106	119
Ice	5/10	St. Line	20	198	180	261	116	144	201
Wet Jennite	20	500'Curve	35	154	222	221	162	219	191
Ice	5/10	500'Curve	20	225	227	284	155	172	215
Wet Jennite	20	Lane Change	35	150	232	221	148	228	178

* *AL = antilock braking system ; LW = locked-wheel braking, best of two stops;CB = controlled braking, best of six stops*

virtually impossible to achieve high levels of braking efficiency under all operating conditions without the use of load sensitive brake proportioning systems and/or antilock braking systems. Because these systems (which add both cost and complexity) are not typically incorporated into heavy vehicles in this country, their stopping capability is generally poor when compared to that of passenger cars.

Driver Considerations. Radlinski presented two areas for consideration for drivers of heavy vehicles—driver's efforts in controlling or modulating the braking system in an emergency situation without locking wheels and losing stability and downhill braking. When a driver is in an emergency braking situation, the only way to be certain that the towed unit is under control during hard braking is to look in the vehicle's mirrors, which is not a desirable situation. Two approaches are used for downhill braking: constant speed/constant drag and snubbing.

In the constant speed/constant drag approach, the driver maintains a constant light drag on the brakes on the down grade. In the snubbing approach, the vehicle speed is allowed to increase with the brakes released and then the brakes are applied at a moderate level to slow the vehicle to a lower speed when the brakes are released. The brakes on/brakes off cycle is repeated on the down grade. Radlinski commented that many driver training manuals recommend the constant drag method, yet many braking experts feel that the snubbing method produces lower, more balanced braking temperatures, and that little data are presented in the literature to support either method as the better approach.

Vehicle Braking Performance. The performance data presented by Radlinski was for vehicles in their newly manufactured condition or "older" vehicles if they are well maintained and equipped with original (or equivalent) braking system components. Radlinski noted that because modulated braking produces more meaningful results in terms of a vehicle's ability to stop safely, most testing is conducted for those conditions. He also noted that because panic stops are hazardous even on the test track, the limited amount of panic stop testing that has been conducted by NHTSA has been done at relatively low speeds. Figure B-7 shows locked wheel braking distance from 30 mph on dry pavement for three vehicle types. The figure indicates that loading does not appear to affect the passenger car but does lengthen the braking distances of the truck and tractor trailer.

This increase appears to be caused by two factors—the sensitivity of the heavy vehicle tires to loading, and the fact that the heavy vehicles did not have sufficient torque on the steering axle brakes to lock the wheels. Loaded trucks and tractor/trailers required between 11 and 17 percent longer braking distances than they did in unloaded conditions. When compared to passenger cars, the heavy vehicles required between 50 and 64 percent longer distances to stop. One cause of the difference is the lower friction coefficient of heavy vehicle tires compared with the passenger car tires.

Figure B-8 shows the braking distances from 60 mph on dry pavement for various air-braked vehicles using a modulated braking technique. Also shown for reference is the braking distance of a typical passenger car under similar conditions. Note that all heavy duty vehicles require longer braking distances than the passenger car.

Buses braking distances are relatively short because their long wheelbase and relatively low center of gravity results in minimal weight transfer during braking. Because the empty versus loaded weight distribution is not significantly different, designers can achieve good brake balance over the range of operating conditions. Radlinski noted that loaded tractor trailers perform reasonably well because they are optimized for the loaded condition; however, loaded trucks do not perform as well as loaded tractor trailers due to the fact that they experience more weight transfer to the front axle, but have relatively small front brakes.

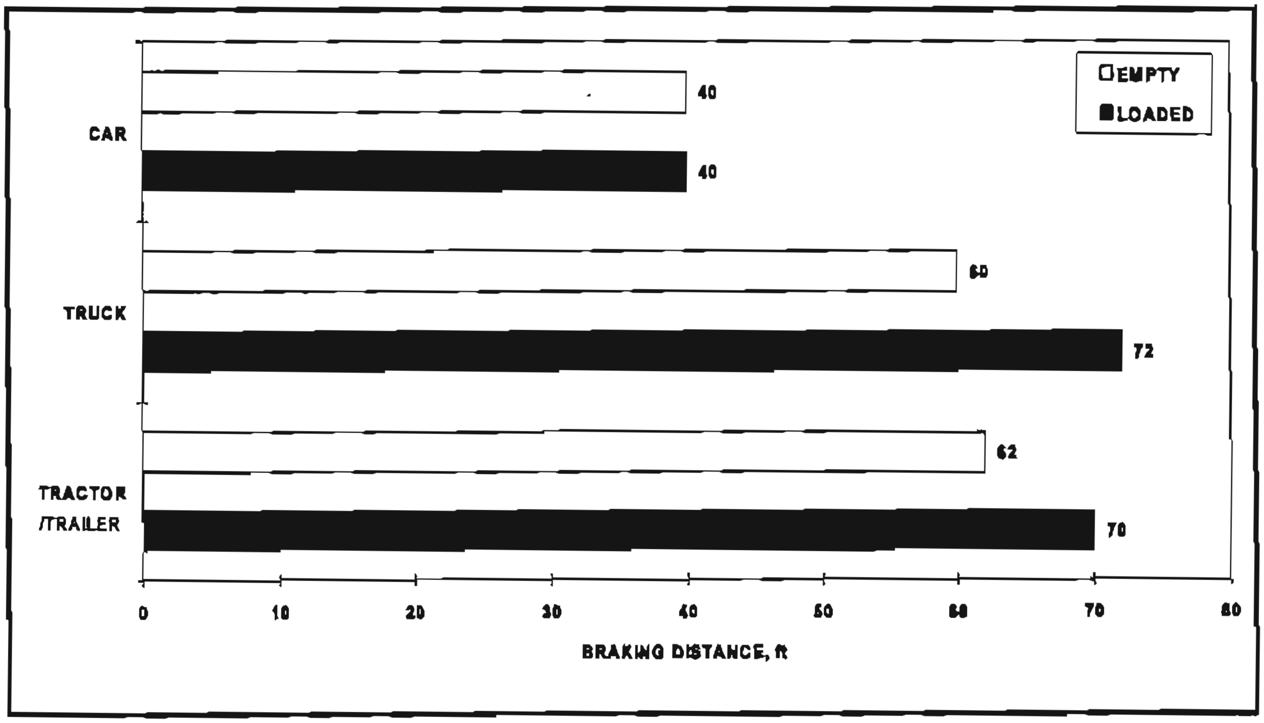

FIGURE B-7. Locked-Wheel Braking Distances From 30 mph on Dry Pavement (*6*).

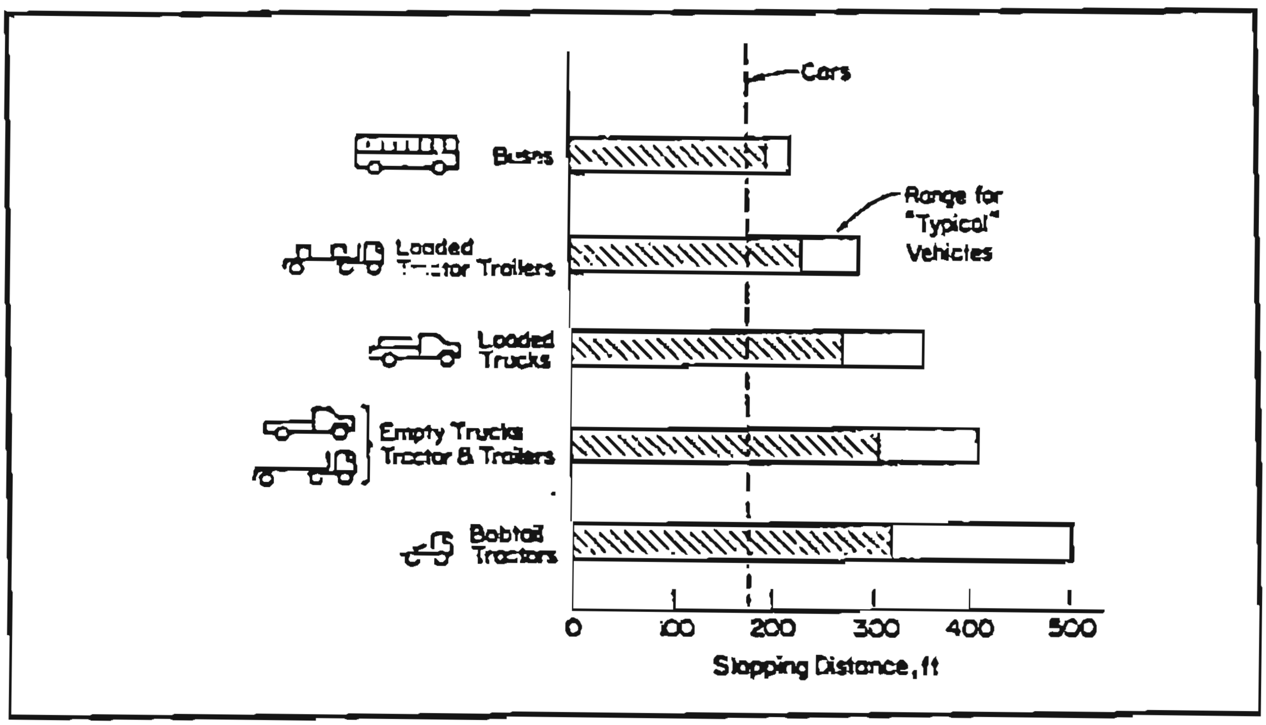

FIGURE B-8. Relative Stopping Capability of Heavy Duty Vehicles From 60 mph on Dry Pavement (*6,23*).

Braking performance of empty vehicles, particularly bobtail tractors, is relatively poor. This result is due to the fact that their braking systems which are sized for loaded conditions produce too much braking at the rear (or trailer) axles when the load is removed. This situation results in premature lockup and corresponding loss of lateral stability capability at the light axle(s) causing the vehicle to become unstable at relatively low decelerations.

Hydraulically-braked trucks were found to perform somewhat better than similarly equipped air-braked trucks. Figure B-9 shows the relative performance of typical trucks with air and hydraulic brakes. Performance of the hydraulically braked vehicles is better because they are typically designed with higher torque front brakes and achieve better braking force distribution, particularly when empty.

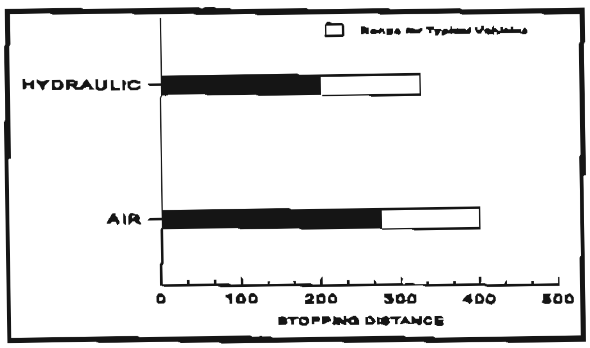

FIGURE B-9. Relative Performance of Trucks Equipped with Air and Hydraulic Brakes—60 mph on Dry Pavement (*6*).

Light Trucks and Cars, Flick and Radlinski (1988)

Due to interest in light truck braking performance as it relates to that of passenger cars, Flick and Radlinski tested 13 light trucks and compared the results to those from 19 passenger cars tested by the same procedure (*24*). These tests were performed in the 1980s at the NHTSA's Vehicle Research and Test Center located at the Transportation Research Center of Ohio. The vehicles selected for the test were not statistically representative of the true passenger car fleet, but did represent a range of vehicle sizes and configurations.

Figure B-10 shows the comparison of the results of the full service brake system tests. The ends of the bars indicate the 95 percent confidence limits in vehicle performance and the line inside the bar shows the average for all of the vehicles. The figure illustrates results at two different loads, two initial speeds (50 and 100 km/h), and two coefficients of friction (Hi-Co and Lo-Co). The Hi-Co surface is concrete with a nominal ASTM skid number of 81 while the Lo-Co surface is wet sealed asphalt with a skid number of 20. Figure B-10 shows that the mean performance of the car and pickup truck samples is less than 3 meters. For both the cars and the trucks, braking performance at light loading is better than at GVWR on both the high and low friction surfaces.

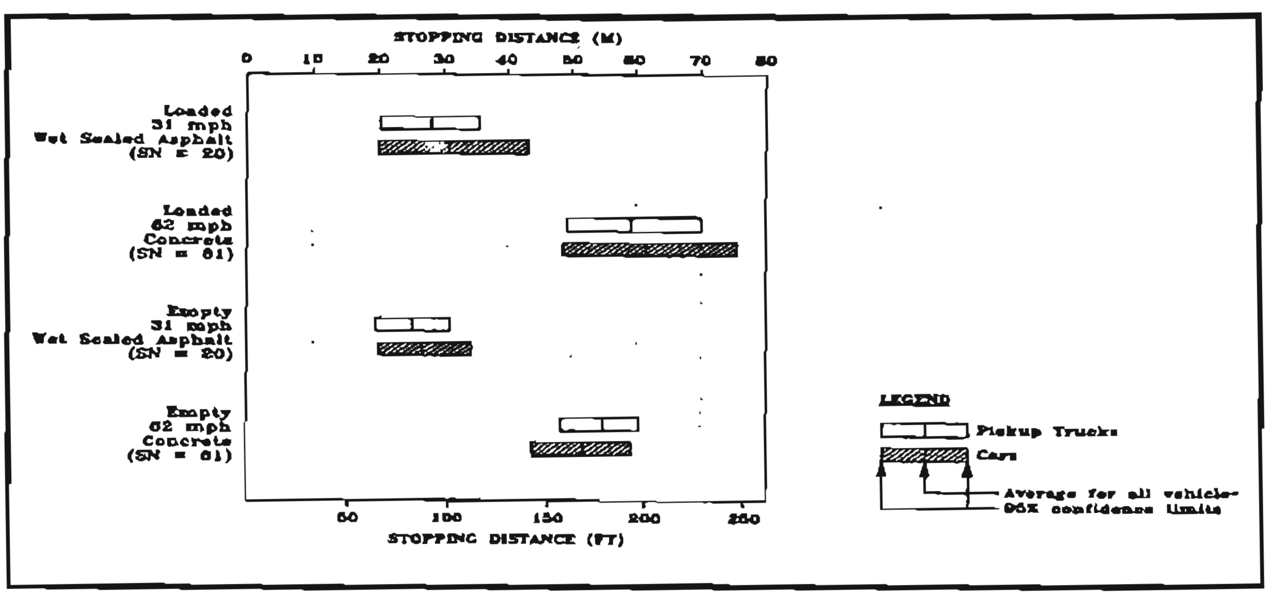

FIGURE B-10. Braking Tests on Passenger Cars and Light Trucks (*23*).

Heavy Truck Braking, Harwood et al. (1990)

Harwood et al. reported on a limited set of NHTSA braking tests that were performed for use in their study. Table B-13 presents the results from these tests. The tests were conducted using straight-line stops on a wet, polished concrete surface with a SN_{40} of about 30. The trucks were tested empty and with radial tires in good condition. The reported results represent the shortest braking distance in a sequence of six tests so as to minimize the influence of driver factors and to represent the best available estimate of the vehicle braking capability (i.e., 100 percent driver control efficiency). All tests were done with the same driver on the same surface.

Stopping Distances of 1988 Heavy Vehicles, Flick (1990)

Flick reported on tests to determine straight-line stopping capability of two single unit trucks and six truck tractors (*25*). The single unit trucks were tested empty and fully loaded while the truck tractors were tested bobtail, with an empty trailer, and with a fully loaded trailer. Tests were conducted to determine the effect of automatic front axle limiting valves (ALV), bobtail proportioning systems (BPS), and antilock braking systems (ABS). All tests were straight-line stops from 60 mph on a dry concrete surface (nominal dry skid number was 81). Table B-14 presents the results from these tests.

The results of the single unit truck tests showed that the loaded braking distances were significantly shorter than the empty braking distances. The braking distances with the antilock brakes operational were shorter than the base vehicle braking distance in both load configurations; however, the percentage decrease in braking distance for the empty vehicle was larger than that for the loaded vehicle. Performance of the bobtail with the bobtail proportioning system was significantly better than the performance of the base vehicle and an even larger improvement was seen with the antilock system. Automatic limiting valves generally increased the braking distance since they reduced the pressure applied to the front brakes in situations where front brake force was already too low.

With empty trailers, combination vehicle braking distances for the base vehicles was shorter than for the bobtail case. Those tractors with antilock brakes showed significant improvements. The automatic limiting valve had a negative effect on the braking performance of three of the tractors/empty trailer combinations, but essentially no effect on the other three combination vehicles. The combination vehicle tests with loaded trailers stopped shorter than the combination vehicles with empty trailers. The two tractors with ABS did not show a significant improvement in their braking distances with loaded trailers because brake balance was near optimum at this load condition.

Light Vehicle ABS Performance Evaluation, Hiltner et al. (1991).

Hiltner et al. reported on tests on ten vehicles that evaluated the improvements in braking performance and vehicle control resulting from adding an antilock braking system (seven passenger cars, two light trucks, and one van) (*26*). The vehicles were tested empty and loaded on different surfaces and speeds. Eight of the vehicles had all-wheel antilock brake systems and two vehicles had rear-wheel only systems. The test series was intended to evaluate the benefits of ABS to an individual vehicle rather than to compare across vehicles. Table B-15 lists the results from the locked-wheel braking tests on wet polished concrete (SN=28) for an "empty" vehicle (driver and instrumentation only) braking in a straight line. Results for other combinations of conditions are included in the report.

Each vehicle's stability improved during braking. Vehicles were more likely to spin because the back wheels locked during braking; (i.e. without the ABS). The all-wheel antilock system improved directional control, but the rear-wheel antilock system did not because the front wheels locked. Braking distances with the all-wheel system were shortened on most hard surfaces. On wet or dry surfaces with high friction values, the difference was relatively small or negligible. On wet highly polished concrete, improvements of 25 percent were observed. Over 50 percent improvements were observed on wet Jennite (sealed asphalt) with the all-wheel systems. The braking distances with rear-wheel systems were not shortened and actually increased in panic situations. All of these results were presumed to be the minimum improvement that ABS could provide because drivers were trained professionals. A typical driver could not be expected to perform as well without antilock brakes and would, therefore , have greater improvements.

Stopping Distance Tests for Two Types of Truck Tire, Tielking and Pezoldt (1992).

Research funded by the AAA Foundation for Traffic Safety studied the impacts of wide base radial truck tires on highway safety (*27*). A portion of the study measured braking distances for vehicles with either conventional dual radial tires or wide base single tires. For comparison, the researchers tested a 1979 Pontiac Grand Am. Table B-16 shows the mean braking distances for the unloaded, wet pavement, locked-wheel tests using five different vehicle types. Table B-17 lists the findings for additional tests conducted using only Vehicle 1 (a long-nose conventional tractor, tank trailer). Mean braking distances for the modulated braking tests are listed in Table B-18. The data presented in Tables B-16 to B-18 were used to compare the stopping distance of vehicles with dual tires to vehicles with wide single tires; therefore, the relative differences were more important than the measured distances.

TABLE B-13. Summary of NHTSA Braking Test on Wet Polished Pavements (5).

Vehicle Type	Vehicle Make and Model	Test Conditions	Braking Distance (ft)		Deceleration (g)	
			30 mph	40 mph	30 mph	40 mph
Passenger Car	Buick Electra	No ABS	66	138	0.45	0.39
		With ABS	71	124	0.42	0.43
Pickup truck	Ford 150	No ABS	72	109	0.42	0.49
Single-unit truck	IH 1954	No ABS	102	180	0.29	0.30
Tractor-semitrailer truck (2S1)	Ford/Great Dane (27 ft)	No ABS/AVL	167	339	0.18	0.16
		With ALV	209	403	0.14	0.13
Tractor-semitrailer truck (3S2)	IH/Fontaine (42 ft)	No ABS/AVL	167	352	0.18	0.15
		With ALV	187	384	0.16	0.14
	Freightliner/Fontaine (42 ft)	No ABS/AVL	167	337	0.18	0.16
		With ALV	198	363	0.15	0.15
		With ABS	100	188	0.30	0.28
	Freightliner/Trailmobile (40 ft)	No ABS/AVL	148	282	0.20	0.19
		With ALV	147	302	0.20	0.18
Double-trailer truck (2S1F2)	Peterbilt/Great Dane (27 ft)	No ABS/ALV	115	225	0.26	0.24
		With ALV	142	283	0.21	0.19
Bobtail tractor (2-axle)	Ford	No ABS/ALV	178	336	0.17	0.16
		With ALV	296	474	0.10	0.11
	Peterbuilt	No ABS/ALV	159	268	0.19	0.20
		With ALV	190	347	0.16	0.15
	Freightliner	No ABS/AVL	167	287	0.18	0.19
		With ALV	195	317	0.15	0.17
		With ABS	90	176	0.33	0.30

Notes: Straight-line, controlled braking on wet polished concrete (SN=28) for an empty vehicle.
ABS = antilock brake system
ALV = automatic limiting value for front-axle brakes

TABLE B-14. Braking Distances of Heavy Vehicles on Dry Pavements (*25*).

Vehicle	Base	With ABS	With ALV	With BPS
Single Unit Truck, Empty				
Ford 4X2	375	na	412	na
Freightliner 6X4	438	233	456	na
Single Unit Truck, Loaded				
Ford 4X2	272	na	na	na
Freightliner 6X4	307	282	na	na
Bobtail Tractor				
Ford 4X2	375	na	412	286
Freightliner 6X4	438	233	456	359
International 6X4	356	na	348	275
Peterbilt 4X2	350	na	414	356
Volvo White 6X4	333	248	353	291
Volvo White 4X2	463	na	531	345
Empty Tractor/Trailer				
Ford 4X2	263	na	300	na
Freightliner 6X4	319	225	322	na
International 6X4	260	na	285	na
Peterbilt 4X2	282	na	287	na
Volvo White 6X4	282	226	279	na
Volvo White 4X2	301	na	316	na
Loaded Tractor/Trailer				
Ford 4X2	230	na	na	na
Freightliner 6X4	266	256	na	na
International 6X4	261	na	na	na
Peterbilt 4X2	273	na	na	na
Volvo White 6X4	253	262	na	na
Volvo White 4X2	261	na	na	na

Note: Braking distances are in feet.

TABLE B-15. Braking Distance for Vehicles With and Without ABS on Wet Pavements (*26*).

Vehicle	Speed	Braking Distance (ft)		Percent Improvement
		with ABS	w/o ABS	
Toyota Supra	35	63	79	20.3
	50	131	198	33.8
Acura Legend	35	75	84	10.7
	50	152	196	22.4
GMC Safari GT Mini Van	35	83	90	7.8
	50	175	197	11.2
Cadillac Brougham	35	72	98	26.5
	50	144	244	41.0
Chrysler Imperial	35	78	98	20.4
	50	164	221	25.8
Pontiac Grand Prix SE	35	78	87	10.3
	50	151	202	25.2
Ford F-150 Pickup Truck	35	100	99	-1.0
	50	228	236	3.4
Mazda B2200 Pickup Truck	35	105	94	-11.7
	50	239	241	0.8
Oldsmobile Cutlass Calais I	35	76	90	15.6
	50	160	225	28.9
Buick Electra	35	78	96	18.8
	50	220	323	31.9

Braking conditions: straight-line, locked-wheel braking on wet highly polished concrete (SN=28), test weight was empty (driver and instrumentation only), average of three stops.

TABLE B-16. Braking Distances for Locked-Wheel Braking From the AAA Study (*27*).

Speed	Vehicle 1 Long-nose conventional tractor trailer		Vehicle 2 Cab-over-engine, van trailer		Vehicle 3 Cab-over-engine, flatbed trailer		Vehicle 3 Cab-over-engine, (bobtail configuration)		1979 Pontiac Grand Am
	Dual Tires	Wide Base	Dual Tires	Wide Base	Dual Tires	Wide Base	Dual Tires	Wide Base	
Unloaded, wet pavement, locked-wheel									
20	52.9	53.3	54.1	55.6	56.4	58.4	94.4	108.2	30
30	119.3	121.5	121.6	115.3	128.5	132.6	223.4	245.5	75
40	224.8	242.4	218.9	224.6	242.1	251.4	394.2	434.4	135
45					313.0	332.9			

Note: The results shown for Vehicle 3 are an average for the Goodyear and Bridgestone tires in the AAA report.

TABLE B-17. Braking Distances for Other Vehicle 1 Tests From the AAA Study (*27*).

Speed	Vehicle 1 - Long-nose conventional tractor, tank trailer	
	Dual Tires	Wide Base
Loaded, wet pavement, locked-wheel		
20	—	47.2
30	94.4	96.6
40	167.1	183.0
Unloaded, dry pavement, locked-wheel		
40	146.6	154.9

Note: Braking distances are in feet.

TABLE B-18. Braking Distances for Modulated Braking Tests From the AAA Study (*27*).

Pavement	Load	Tire	Vehicle 1		Vehicle 2	
			Average	Range	Average	Range
Modulated braking from 40 mph						
Dry	No Load	Dual Tires	181	164-190	204	179-224
		Wide Base	181	175-187	206	184-233
Wet	Loaded	Dual Tires	283	271-301		
		Wide Base	261	222-309		
Wet	No Load	Dual Tires			256	236-273
		Wide Base			287	256-319

Note: Braking distances are in feet.

The tires used in the tests were new. The steering axle brakes on the trucks were disabled because available information indicated that only about half of randomly surveyed heavy vehicles have operative front axle brakes. The majority of the tests for the AAA study were conducted on wet pavement; however, a limited number of tests were performed on dry pavement. Most tests were conducted under straight-line, locked-wheel conditions. All testing was performed on an asphaltic aggregate pavement. Approximate friction numbers for the test pavement at 40 mph was 0.38 for the wet condition and 0.72 for the dry condition. The measured braking distances were adjusted to account for the small differences between the intended initial speed and the actual measured speed when the brakes were applied.

Heavy Truck Round Robin Brake Test, Kempf (1992)

A 1992 paper by Kempf described the *Heavy Truck Round Robin Brake Tests* that involved three test vehicles at ten different proving grounds (*28*). The purpose of the program was to evaluate the practicality and repeatability of the test procedure proposed by the ABS Test Procedure Task Force, a subunit of the Heavy Truck Subcommittee of the Motor Vehicle Safety Research Advisory Committee, and to compare surface friction characteristics of dry high friction and wet low friction test surfaces at different proving grounds. The dry high friction test is performed in a straight line; the wet low friction test involves braking in a 500-foot radius curve. At each site, a minimum of six straight-line stops (from 60 mph) were performed on the dry high friction asphalt or concrete surface. A minimum of three wet low friction tests were performed at seven of the ten sites from 25 mph, 30 mph, and/or at maximum speed.

Both the high and low friction tests were conducted at two weight conditions: bobtail (empty) and half (partial) load. For the bobtail condition, the vehicle was at curb weight plus driver and instrumentation. For the half load condition, the tractor was coupled to a 28-foot unbraked control flatbed trailer and the trailer was loaded until one-half of the tractor's driver axle(s) gross axle weight ratio was obtained. Kempf concluded that the proposed test procedure appeared to yield repeatable results. The braking distances of the test vehicles remained relatively constant on the dry high friction surface. The range of average braking distances from 60 mph is listed in Table B-19. There was a large variation in vehicle braking distance on the wet low friction surface. Table B-19 lists the range of average braking distance from 25 mph.

TABLE B-19. Heavy Truck Round Robin Braking Test Results (*28*).

	Dry High friction 60 mph	Wet Low friction 25 mph
Bobtail	206 to 218 ft	76 to 165 ft
Half Load	255 to 280 ft	91 to 197 ft

COMPARISON OF VARIOUS BRAKING STUDIES

There are many different factors that influence braking distances. Because these factors typically vary between different studies, comparison of the results is very difficult. Care must be exercised in comparing results so that conclusions are not falsely drawn. Data from actual vehicle braking tests are generally at lower speeds (less than 40 mph) because of the hazardous nature of attempting to stop a truck or a passenger car at high speeds, especially when using poor conditions such as worn tires and stopping on wet, poor quality pavements. This concern results in limited or no data available for actual braking distances in these situations.

Tables B-20 to B-21 list the factors associated with the simulation and field tests studies, respectively, reviewed for this report. These tables demonstrate that several variables differ between what may initially appear as similar studies. To provide another method to illustrate the findings from the different studies in a more comparable format, Figures B-11 to B-16 were generated. These figures illustrate the braking distances for cars or trucks using locked-wheel, controlled, or antilock brakes stopping methods. On each of the figures, Curve A always represents the AASHTO braking distances (*2*). This curve serves as a "reference line" to compare the findings from different studies to what is currently used in design. Not all of the findings discussed earlier in this report are presented on these figures. Generally, only those findings that included assumptions similar to the AASHTO braking assumptions were included. These assumptions include wet pavements, poor pavements, and/or worn tires. The findings from studies using other assumptions were include to illustrate differences when pertinent.

In Figure B-11 (locked-wheel braking for passenger cars), each of the curves is below or near the 1990 AASHTO curve. Actual braking distances for cars and pickup trucks in a controlled braking situation (see Figure B-12) and in an antilock braking situation (see Figure B-13) also fall beneath the 1990 AASHTO curve. Braking distances calculated using the equations developed based on the NCHRP 270 work (see Table B-7), produced curves that are near or above the AASHTO curve (see Figure B-12). One possible explanation for the NCHRP 270 curves being higher than the other data is the effect of assuming a worst case situation for each variable. Only rarely would the combination of an unalerted driver, worn tires, poor brakes, wet pavement, and a 15th-percentile quality pavement occur. Curve B on Figure B-12 demonstrates a worst-case scenario. For passenger cars, the AASHTO assumptions for distances are conservative in the locked wheel and controlled situations (when compared to actual braking tests). The AASHTO assumption is less conservative when compared to the predicted values from the equations in Table B-7.

Braking distances for trucks in the locked-wheel and controlled situations are generally higher than the braking distances assumed by AASHTO (see Figures B-14 and B-15). The braking distances for trucks using antilock brakes are near the 1990 AASHTO curve (see Figure B-16). The incorporation of antilock brake systems into the heavy truck vehicle fleet has the potential of greatly reducing the braking distances of the heavy vehicles.

TABLE B-20. Factors Used in the Simulation Studies Reviewed.

Study	Source of Data	Vehicle Type	Type of Braking	Pavement Quality	Tire Condition	Speeds	Numeric values
AASHTO, 1990 *Green Book* (*2*)	based on studies by Moyer and Shuppe in 1951	Passenger cars	locked-wheel	wet, poor, SN=32	worn	20 to 70 mph	Table B-1
Equations based on NCHRP 270 (*13*)	equations based on literature findings	car or truck	locked- wheel or controlled	user selects SN and mean pavement texture depth	user selects	user selects	Table B-7
NCHRP 270 findings (*13*) (1984)	numerical integration of information from literature	car & truck (unladen)	locked- wheel & controlled	wet, poor (15 percentile)	worn (2/32 tread depth) and new (≥12/32 tread depth)	20 to 80 mph	Figure B-5 (cars) Figure B-6 (trucks)
Harwood et al. (1990) (*5*)	calculated based on 270s work and other assumptions	truck	controlled	wet, poor (same pavement as AASHTO, SN_{40}=32)	good tires	20 to 70 mph	Table B-10
Kulakowski et al. (1992) (*22*)	simulation	truck	controlled	dry and wet, low and high roughness	good	55 mph	Table B-11

TABLE B-21. Factors Used in Field Braking Studies Reviewed.

Study	Source of data	Vehicle type	Type of Braking	Braking Maneuver	Pavement	Speeds	Values
Radlinski & Bell (1986) (*21*)	Braking tests conducted at the NHTSA facility in Ohio	Two-axle truck (empty and loaded)	antilock, locked wheel, and controlled	straight line stop, stop in and turn, and stop in lane change	dry asphalt, wet asphalt, wet polished concrete, wet sealed asphalt, ice	20 to 60 mph	Table B-12
Radlinski (1987) (*6*)	Braking tests conducted at the NHTSA facility in Ohio	car, truck, tractor/trailer (loaded and unloaded)	assumed controlled	assumed straight line	dry	30 mph	Figure B-7
		car, buses, loaded & empty trucks	stable (controlled)	assumed straight line	dry	60 mph	Figure B-8
Flick & Radlinski (1988) (*24*)	Braking tests conducted at the NHTSA facility in Ohio	light trucks and passenger cars (loaded and lightly loaded)	assumed controlled	assumed straight line	high coefficient of friction (SN=81) and low coefficient of friction (SN=20)	31 mph and 62 mph	Figure B-10
Harwood et al. (1990) (5)	NHTSA's braking tests conducted for FHWA study	pc, pickup, single-unit, 3S3, bobtail (empty)	controlled, antilock, and ALV	straight line	wet, polished concrete (SN_{40}=30)	30 and 40 mph	Table B-13
Flick (1990) (25)	Braking tests conducted at the NHTSA facility	2 single unit trucks, 6 truck tractors	controlled, antilock, ALV, BPV	straight line	dry concrete (SN=81)	60 mph	Table B-14
Hiltner et al. (1991) (*26*)	Braking tests conducted at the NHTSA facility	7 cars, 2 pickups, 1 van (loaded and empty)	controlled, antilock, and panic	straight line, stop in curves, stop in lane changes	gravel, wet concretes, dry concrete, wet asphalt, wet Jennite, split friction	ranging between 35 and 60 mph	Sample values in Table B-15
Tielking & Pezoldt (1992) (*27*)	Braking tests conducted at TTI	3 tractor/trailers, 1 bobtail, 1 car	controlled and panic	straight line	wet and dry asphaltic aggregate	20,30, 40, and 45 mph	Tables B-16 to B-18
Kempf (1992) (*28*)	Braking tests conducted at ten different facilities	3 tractor/semitrailers	controlled	Straight line, 60 mph, dry high friction surface (friction=.87 to 1.0) 500 feet radius, 25 mph, 30 mph, and/or maximum speed, low friction surface (friction=.29 to .49)			Table B-19

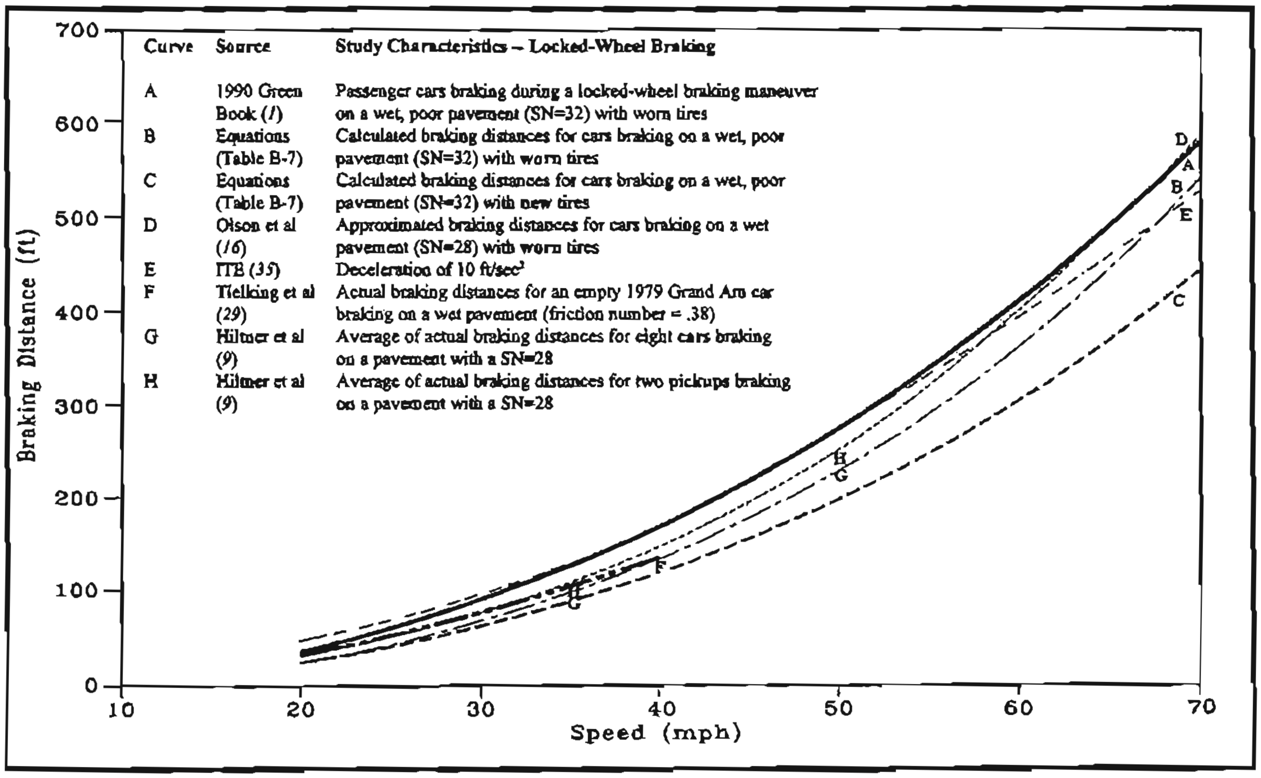

FIGURE B-11. PANIC (Lock-Wheel) Braking for Passenger Cars.

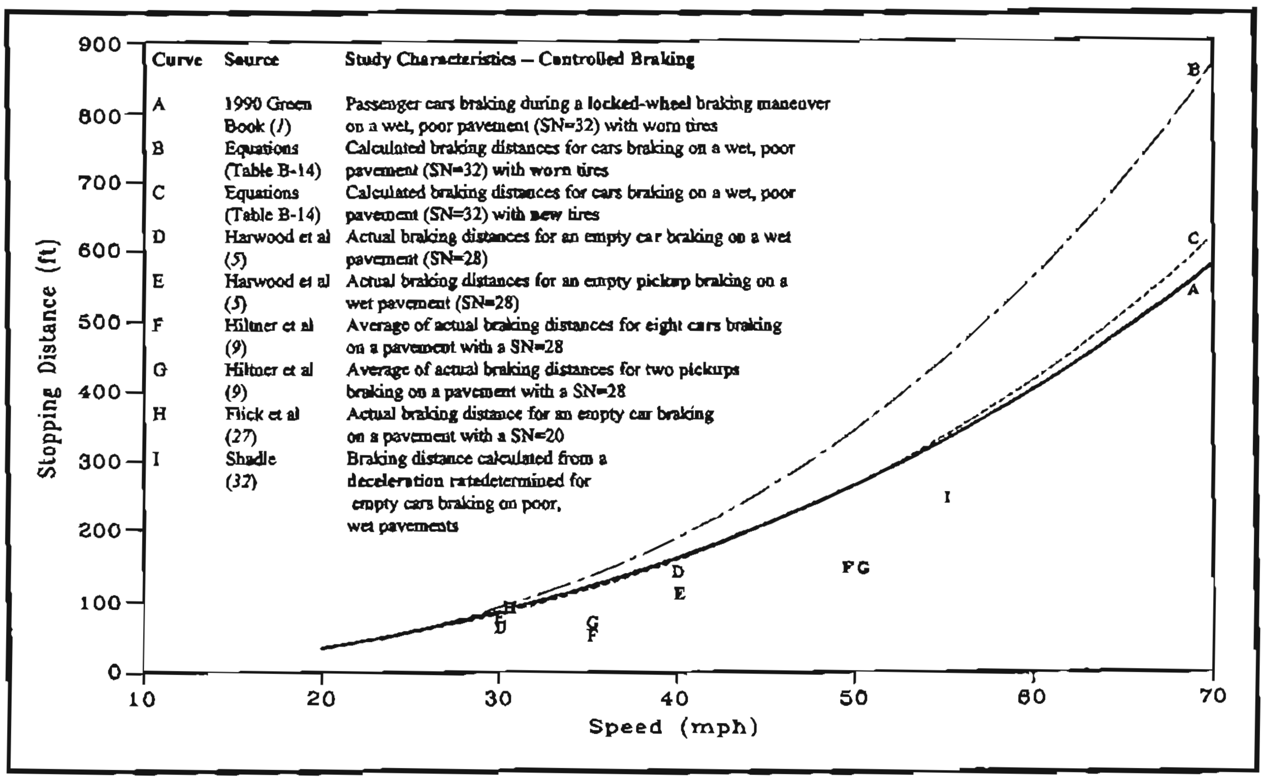

FIGURE B-12. Controlled Braking Distances for Passenger Cars.

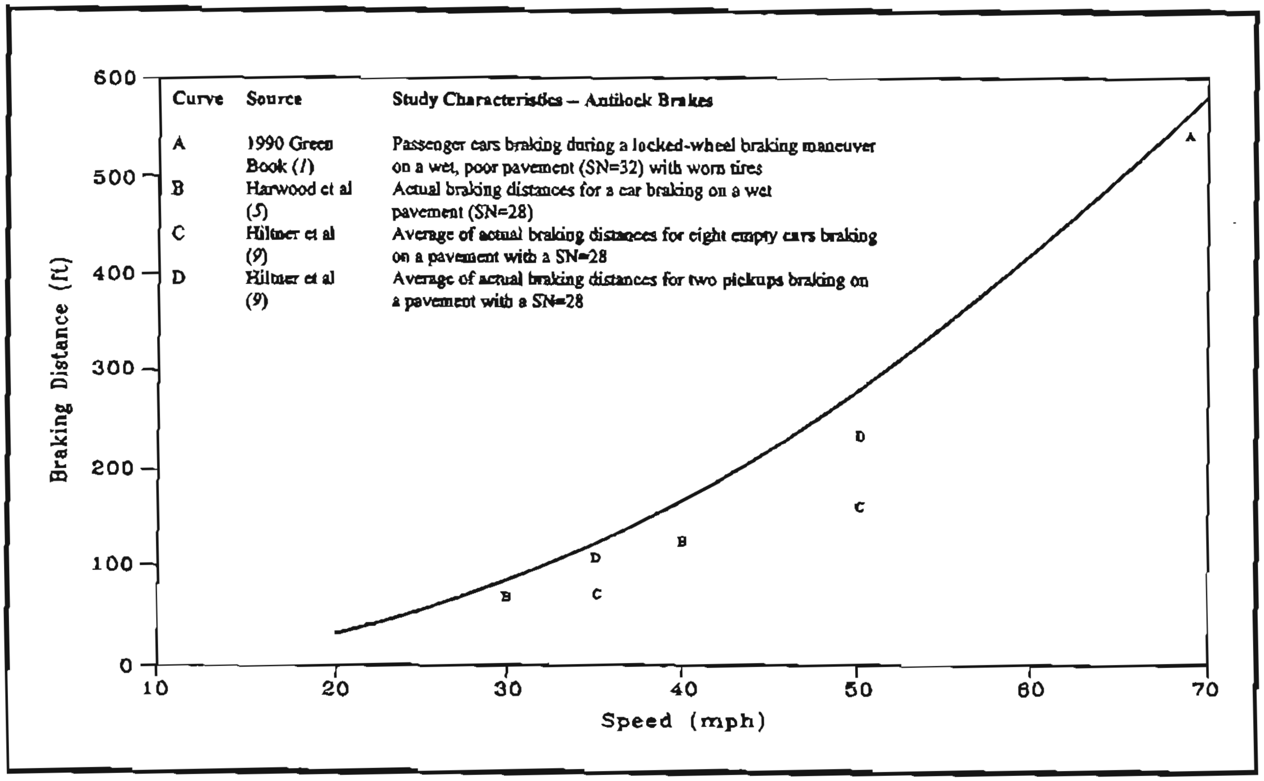

FIGURE B-13. Antilock Braking for Passenger Cars and Pickups.

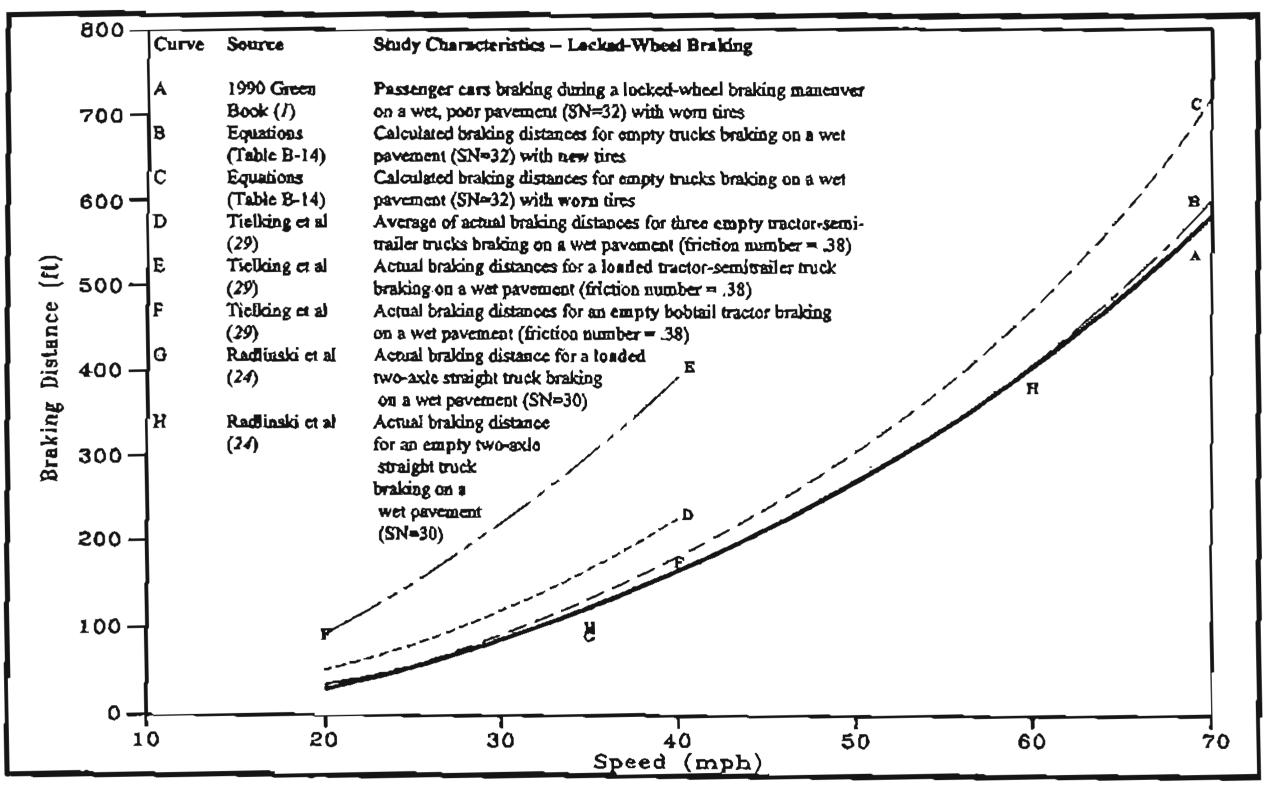

FIGURE B-14. Panic (Locked-Wheel) Braking for Trucks.

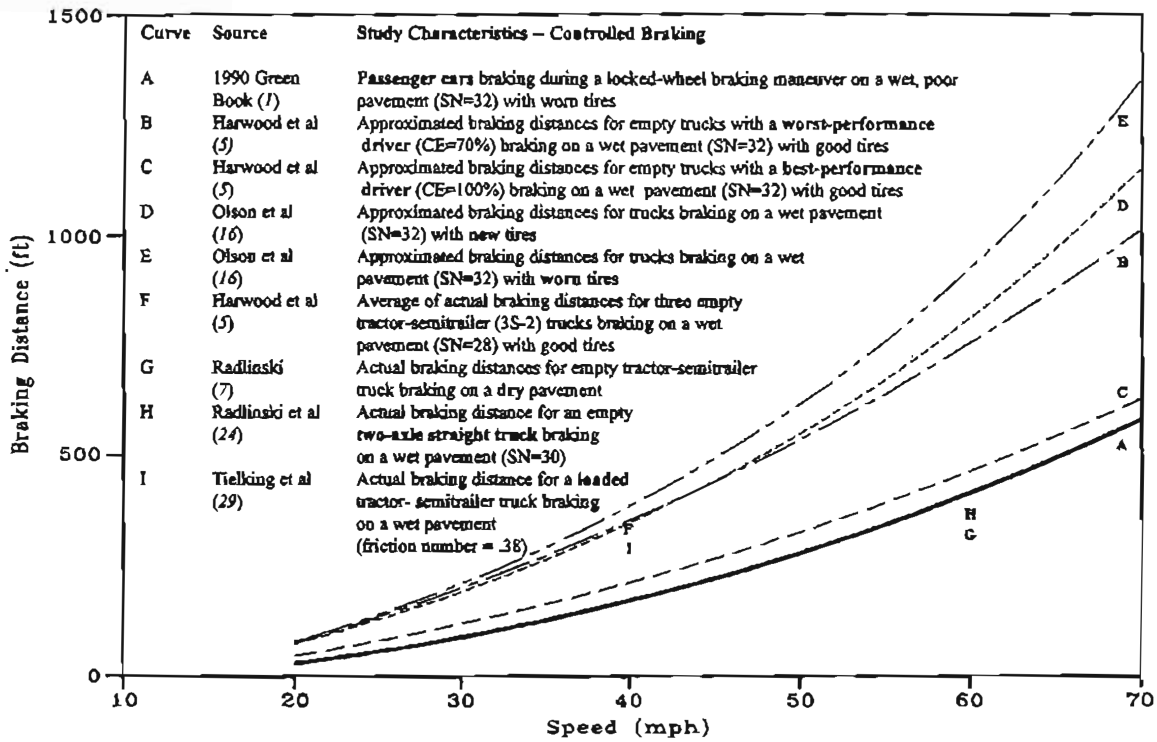

FIGURE B-15. Controlled Braking for Trucks.

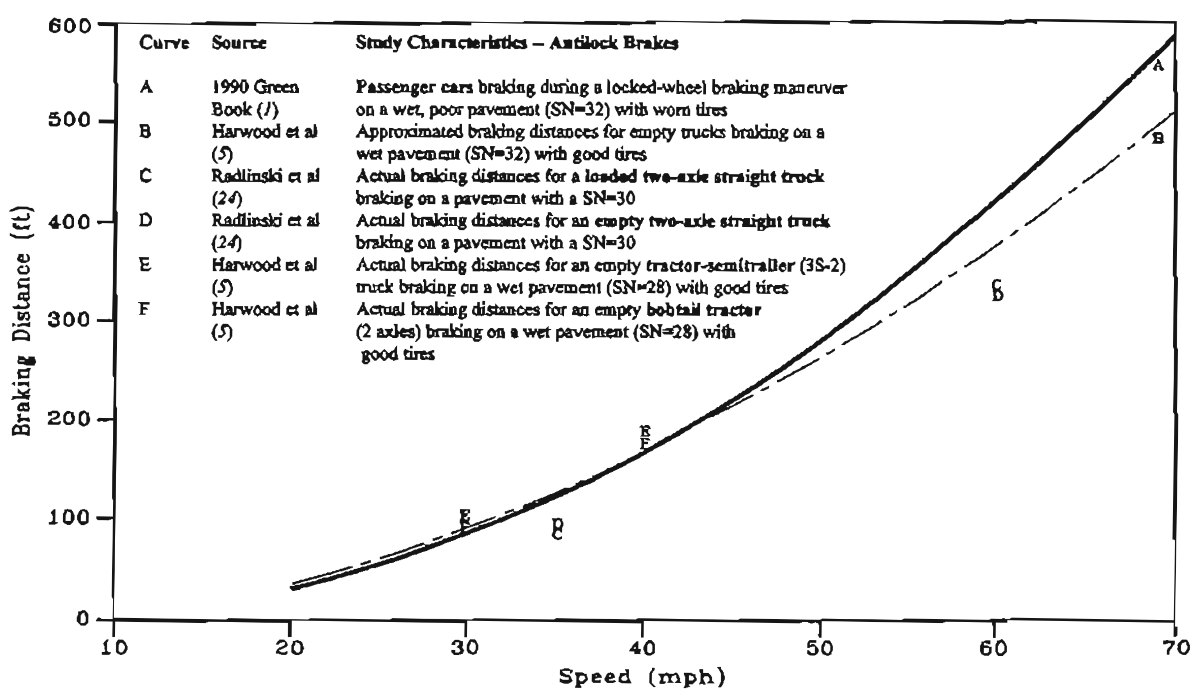

FIGURE B-16. Antilock Braking for Trucks.

PAVEMENT FRICTION CHARACTERISTICS

The braking distance calculation in the AASHTO equation (see Equation 1) requires the use of a friction factor. This factor can be (and has been) estimated using skid number data from actual skid tests. Several studies have argued that a value lower than an "average" or "typical" value should be used in design (*3,4,13*). One study suggested the 15th percentile skid number be used for design (*13*).

To select design a skid number or to use skid data to verify or compare existing assumptions with actual data requires that such data be available. For this study, data from two states and from Strategic Highway Research Program (SHRP) database were obtained. Skid numbers and other relevant data were extracted from the databases and then sorted into functional roadway classes. Cumulative frequencies of the individual skid numbers were developed for each functional class. The cumulative frequencies could then be used to determine the 15th percentile (or any other desirable percentile) skid number for each roadway type. Following are the findings from the Texas, California, and SHRP databases. Also included in the discussion are findings from a report on skid number data of three California districts.

Texas Data

The high, low, and average skid numbers for different highway segments were provided by the Texas Department of Transportation. The skid data represented skidding efforts between 1986 and 1991. The database contained several pieces of information pertinent to the skid number; however, it did not contain data pertainent to the functional class of the segment. Extensive manipulations between the skid number database and the Texas Roadway Inventory Log provided information needed to sort the skid numbers by functional class. The variables and the variables' values used to sort the data into ten different functional classes are listed in Table B-22.

The average skid number, rather than the high or low value, was selected for use in this analysis. If more than one skid number was present for a particular lane (because the road was skid tested two or more times within the five-year period), then only the most current value was used in the evaluation. Cumulative frequency curves were generated for each functional class and are shown in Figures B-17 and B-18. Table B-23 lists the 15th, 50th, and 85th percentile skid numbers and the number of records (or skid numbers) for each functional class. Note that each functional class has a 15th percentile skid number equal to or greater than the skid number of 32 associated with the AASHTO stopping sight distance model.

California Data

The California Department of Transportation (CALTRANS) provided magnetic tapes from their Skid Resistance Inventory (SRI) file. This file includes skid number by section of roadways. Because California generally skid their pavements every two years, the two most recent years of data (July 1990 to June 1992) were used for analysis in the study. The data were sorted into eight functional classes using the variable descriptions listed in Table B-24. Cumulative frequency curves were generated for each functional class. These curves are illustrated in Figures B-19 and B-20. Table 27 lists the 15th, 50th, and 85th percentile skid numbers and the number of records for each functional class.

TABLE B-22. Texas Roadway Inventory Log Variables Used to Determine Roadway Functional Classes.

Functional Class For This Study	Texas Functional Class	Highway Design Type	Number of Lanes	Shoulder Type
Rural Freeway	Interstate	Freeway—zero, one, or two service roads	>2	paved or unpaved
Rural Multilane Divided	na	Boulevard or expressway	>2	paved or unpaved
Rural Multilane Undivided	na	Two-way or one-way traffic	>2	paved or unpaved
Rural Two-Lane High	na	Two-way or one-way traffic	2	paved, concrete brick or block
Rural Two-Lane Low	na	Two-way or one-way traffic	2	earth, unpaved, 6 ft
Urban Freeway	Interstate, Urban Freeway	Freeway—zero, one, or two service roads	>2	paved or unpaved
Urban Multilane Divided	na	Boulevard or expressway	>2	paved or unpaved
Urban Multilane Undivided	na	Two-way or one-way traffic	>2	paved or unpaved
Urban Two-Lane High	na	Two-way or one-way traffic	2	paved, concrete, brick or block
Urban Two-Lane Low	na	Two-way or one-way traffic	2	earth, unpaved, 6 ft

Note: Rural = populations les than 5,000; Urban = populations greater than 5,000

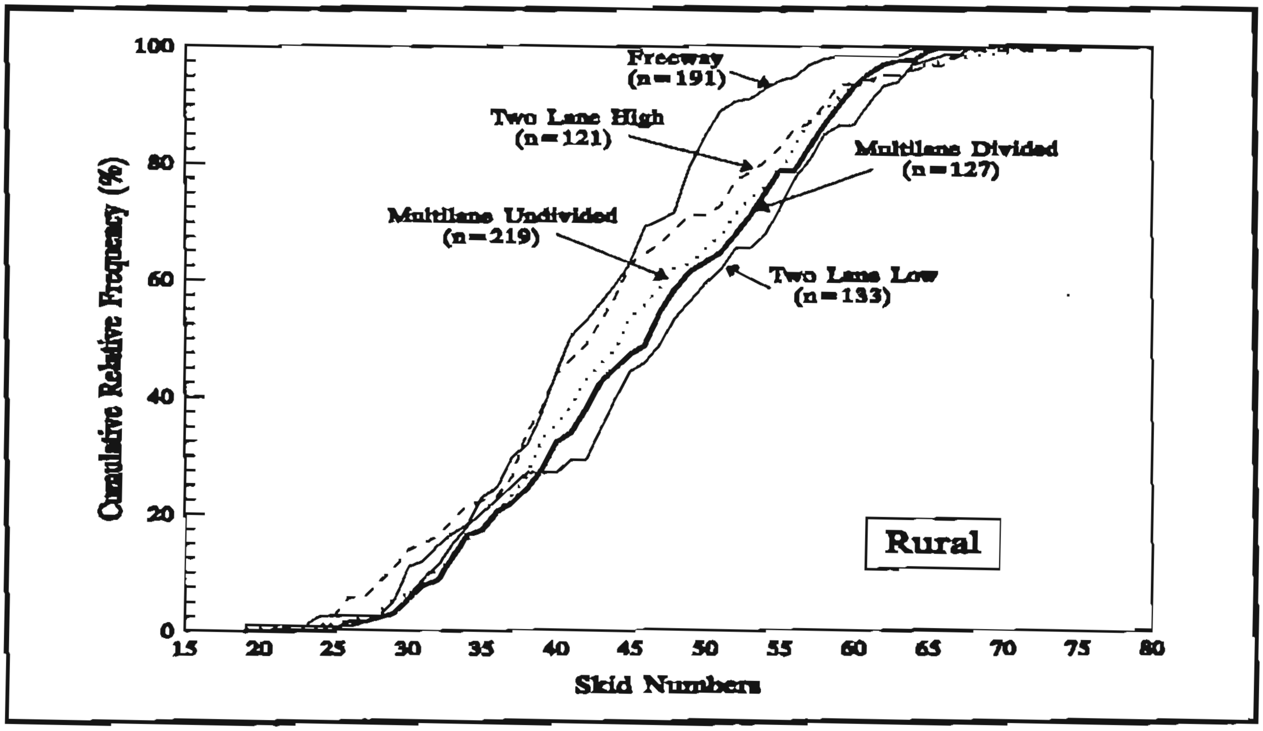

Figure B-17. Cumulative Skid Numbers from Rural Texas Data.

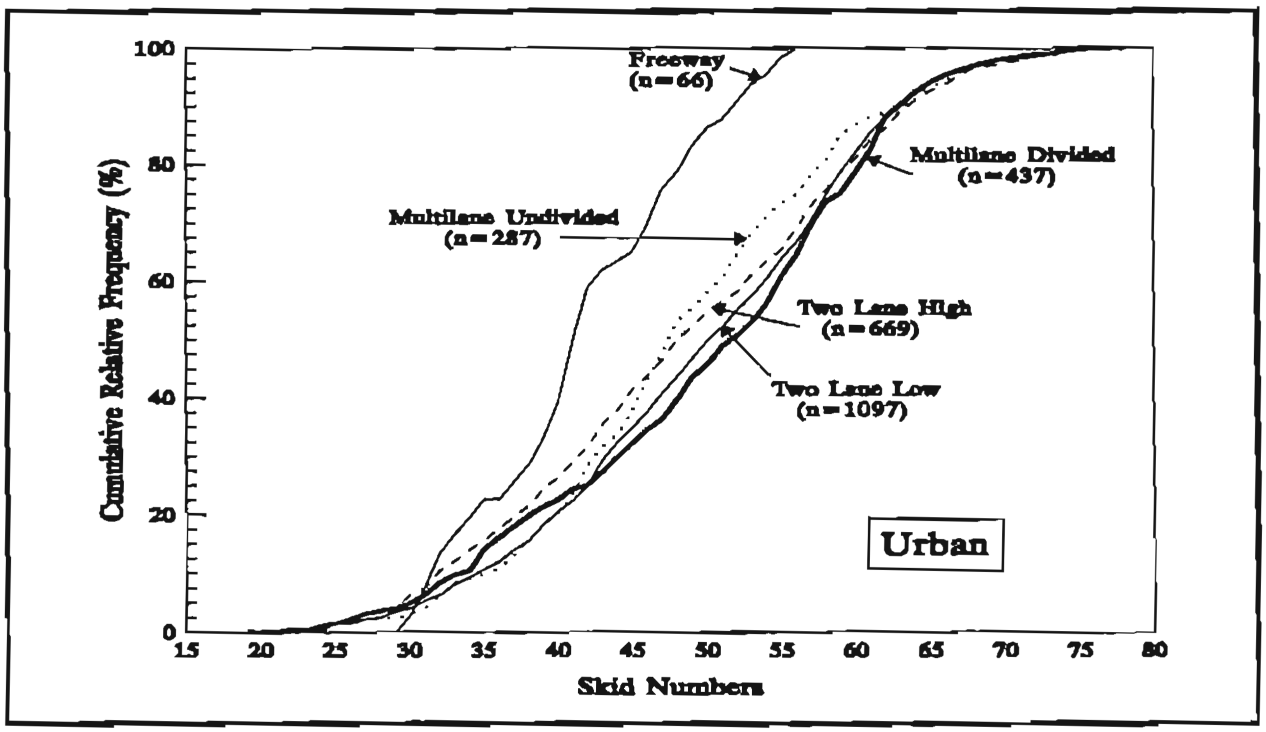

FIGURE B-18. Cumulative Skid Numbers From Urban Texas Data.

TABLE B-23. Skid Numbers From Texas Database.

Functional Class	Percentile			Number of Records
	15th*	50th	85th	
Rural Freeway	33	41	51	191
Rural Multilane Divided	34	47	58	127
Rural Multilane Undivided	34	45	57	219
Rural Two-Lane High	32	43	56	121
Rural Two-Lane Low	33	48	59	133
Urban Freeway	33	41	50	66
Urban Multilane Divided	36	52	62	437
Urban Multilane Undivided	38	48	59	287
Urban Two-Lane High	35	48	62	669
Urban Two-Lane Low	38	51	61	1097

* *15 percent of the pavements have this skid number or worse (or, 85 percent of the pavements have this skid number or better).*

TABLE B-24. Variables Used to Determine Roadway Functional Classes for California Data.

Functional Class For This Project	Rural or Urban	Divided or Undivided	Number of Lanes	Freeway or Expressway
Rural Freeway	Rural	Divided	>1	Freeway or Expressway
Rural Multilane Divided	Rural	Divided	>1	Other than Freeway or Expressway
Rural Multilane Undivided	Rural	Undivided	>1	Other than Freeway or Expressway
Rural Two-Lane Roadway	Rural	Undivided	1	Other than Freeway or Expressway
Urban Freeway	Urban	Divided	>1	Freeway or Expressway
Urban Multilane Divided	Urban	Divided	>1	Other than Freeway or Expressway
Urban Multilane Undivided	Urban	Undivided	>1	Other than Freeway or Expressway
Urban Two-Lane Roadway	Urban	Undivided	1	Other than Freeway or Expressway

TABLE B-25. Skid Numbers From California Database.

Functional Class	Percentile			Number of Records
	15th*	50th	85th	
Rural Freeway	33	42	50	18,990
Rural Multilane Divided	30	41	49	904
Rural Multilane Undivided	32	43	52	1,192
Rural Two-Lane	32	43	51	22,457
Urban Freeway	33	40	47	22,274
Urban Multilane Divided	28	36	44	5,297
Urban Multilane Undivided	27	36	43	1,173
Urban Two-Lane	27	36	45	2,211
All Roadways	32	41	49	74,498

* *15 percent of the pavements have this skid number or worse (or, 85 percent of the pavements have this skid number or better).*

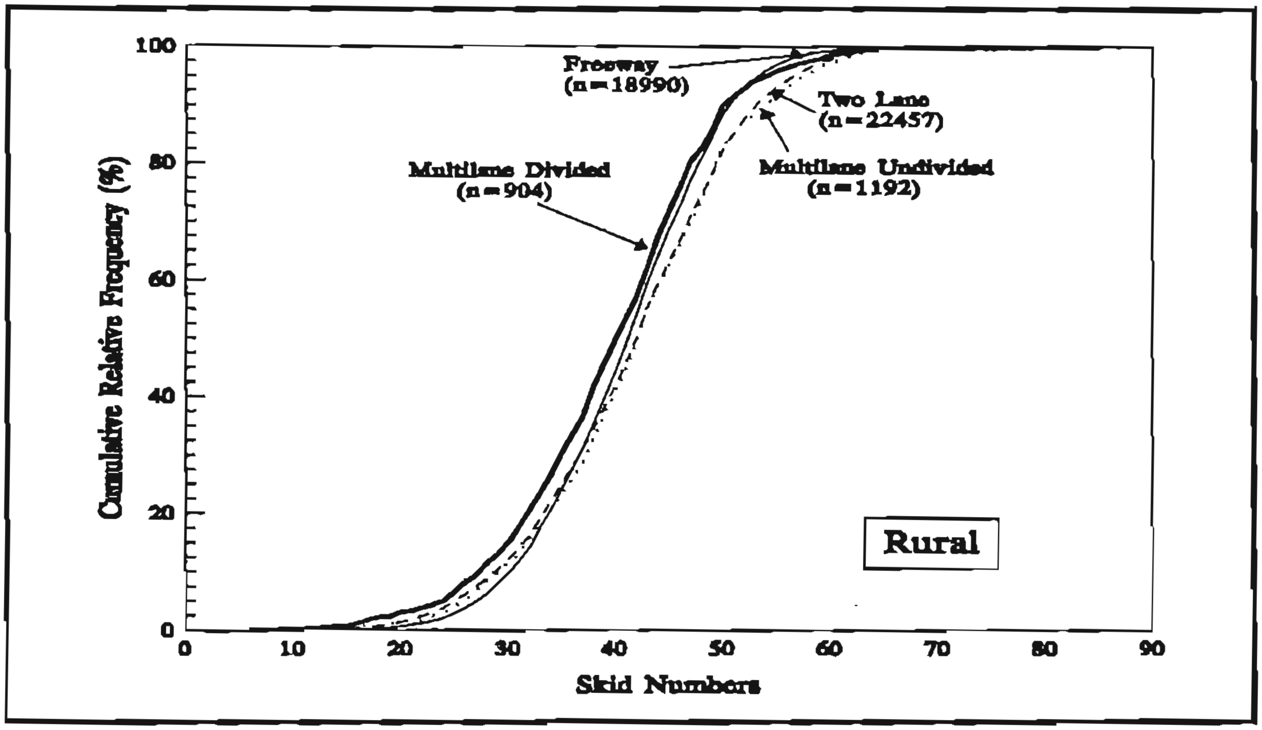

FIGURE B-19. Cumulative Skid Numbers from Rural California Data.

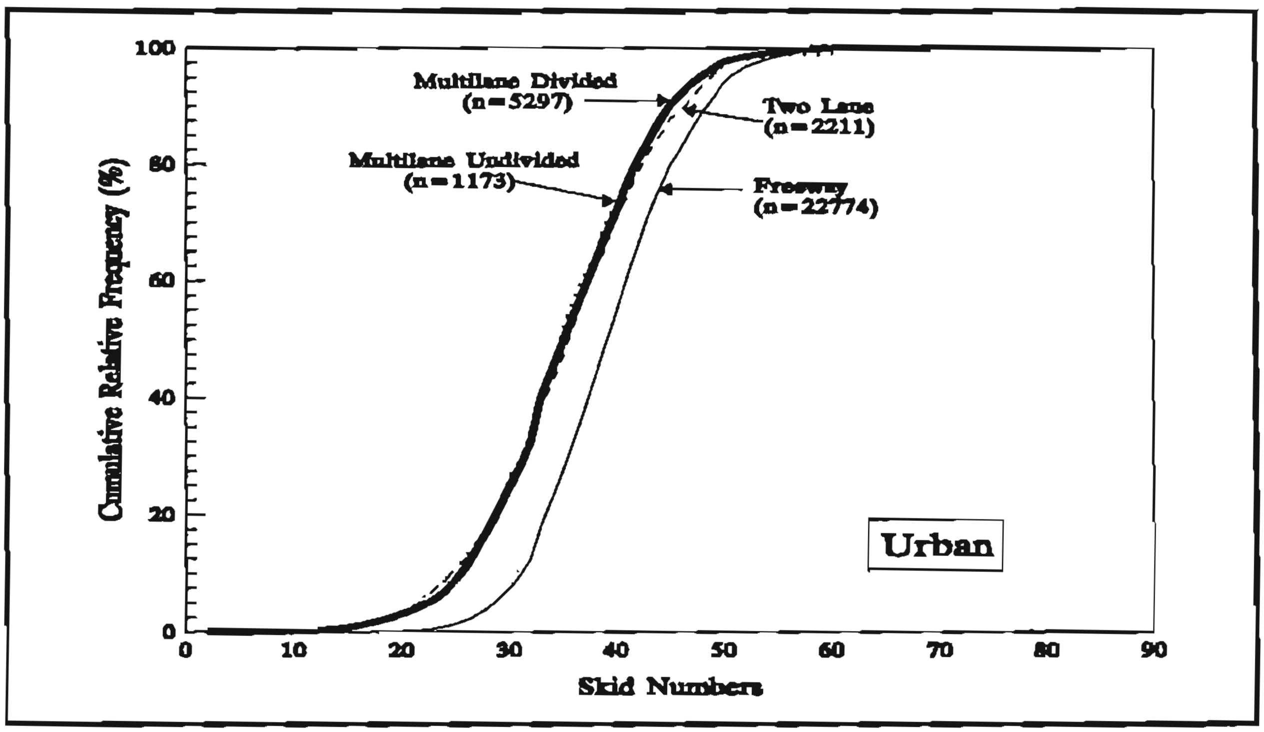

FIGURE B-20. Cumulative Skid Numbers from Urban California Data.

Most of the rural functional classes have 15th percentile skid numbers that are at, or just above, the 32 skid number assumed by AASHTO's stopping sight distance model (the multilane divided is just below). Most of the urban functional classes, however, are below the 32 value. Only the urban freeway class has a higher skid number. The California SRI file also was used in a 1986 study that related skid numbers to wet pavement accident frequency. The findings from that study follow.

In 1986, a FHWA report included a distribution of skid numbers that were used in a study of highway geometrics and wet pavement accident frequency (*30*). Data from three CALTRANS Districts, for a one-year period, were used. Based on a wet pavement accident rate established from average daily traffic, percent wet time, and the number of times a specific skid number occurs for any geometric classification, the following conclusions were made:

- Curves have the highest accident rate followed by weave sections and intersections;
- Accident rates are substantially higher at locations having skid numbers less than 25; and
- The accident rate is nearly constant on pavements with skid numbers greater than 26, but it increases substantially as the skid number decrease from 25 to 17.

Figure B-21 shows the histogram of skid numbers between 17 and 54 for the three districts in California. The skid numbers have a mean of 37 and a standard deviation of 6.6. Table B-26 lists the mean skid number for different pavement types.

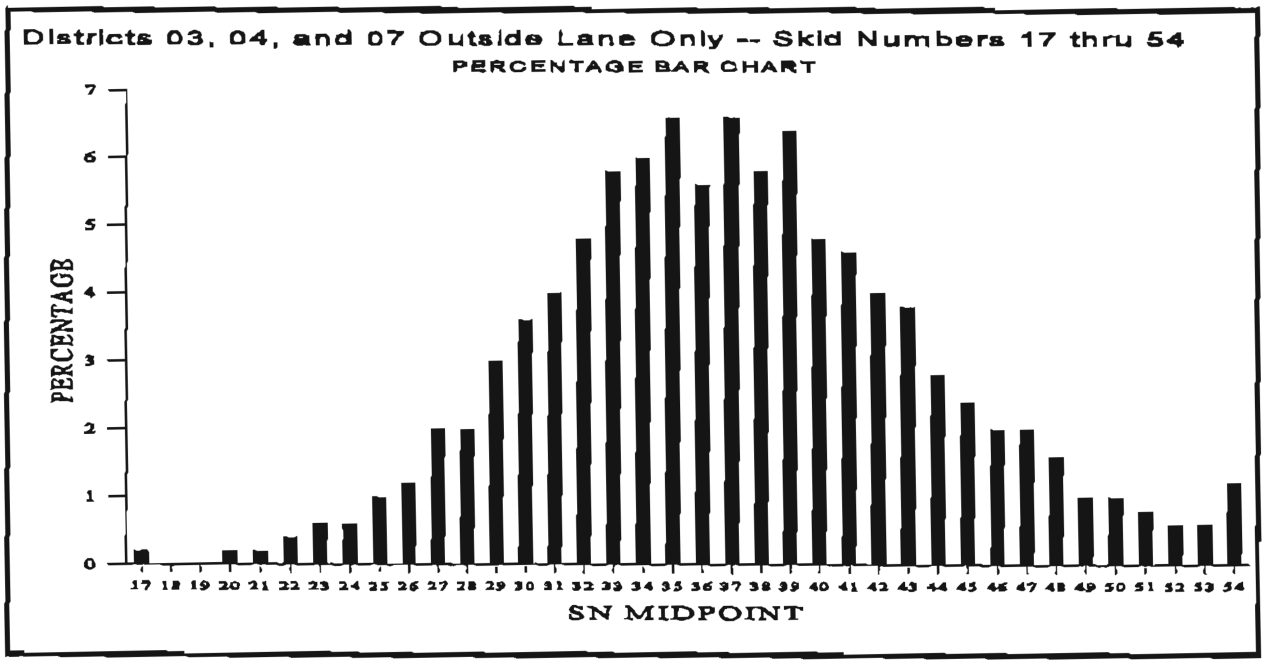

FIGURE B-20. Histogram of Skid Numbers for Three California Districts (*29*).

TABLE B-26. Mean Skid Numbers by Pavement Type for Three California Districts (*29*).

Surface	Average Skid Number	Number of Samples
dense graded asphalt concrete	38	3932
open graded asphalt concrete	37	704
portland cement concrete	36	3103
portland cement concrete--grooved	37	1433
chip seal	44	243
slurry seal	38	48
epoxy	34	16
patch	34	7
other	38	4

National Data

The national data were from the Long-Term Pavement Performance (LTPP) Information Management System (IMS). The LTPP IMS is a database management system developed under SHRP, a five year $150 million research program that concentrated on a short list of high-payoff activities in asphalt, pavement performance, concrete and structures, and highway operations. The LTPP program will collect data on in-service pavement sections throughout the country for a twenty-year period.

Data stored in the IMS are collected from SHRP test sections located throughout the United States and Canada. A variety of information and data is collected for each section including climatic, material properties, traffic loads, friction, and numerous other types of data. The data of primary interest for this project is the skid numbers.

Skid numbers, or friction measurements, are taken at least every two years by state agencies and recorded in the IMS. The skid number, time of day, surface type, vehicle speed, and test method are some of the principal elements stored. Each section in the database is 500 feet in length and the skid tests were all conducted at 40 mph. Two skid numbers were reported for each section, one at the beginning of the section and the other at the end of the section. The values reported in this project reflect an average of these two numbers. Skid data were available for 687 sections. The data were from the following regions: Western, Southern, North Central, and North Atlantic.

Pavement sections studied under the LTPP Program are divided into either General Pavement Studies (GPS) or Specific Pavement Studies (SPS). GPS sections are in-service pavements nominated by state and provincial Department of Transportation officials and selected by SHRP. These pavements generally represent the most common pavement structural designs used nationally and internationally. SPS test sections are specially designed pavement structures chosen to develop a better understanding of the effects on performance of a few targeted factors not adequately covered in the GPS. SPS sections are constructed under the SHRP program to allow for initiation of monitoring of performance from the initial date of construction or opening to traffic. The intent of SPS is to collect more reliable data over the entire life of the section to better calibrate performance prediction equations.

Procedures and standards were established for the collection and recording of data. These procedures are to help guarantee the consistency and the quality of the data collected. Extensive data quality checks are performed throughout the collection and recording process to further ensure the best possible data. The data used in this project has passed the Level 1 (sectional) quality assurance checks. The checks include: random checks of data to ensure correct data transfer, data searches to verify that critical elements are present, and expanded range checks on certain fields to identify data elements values which fall outside an expected range. October 1992 data were used in this analysis.

The skid data were divided into functional classes that would be similar to the classes used to evaluate the Texas data. Table B-27 lists the variables in the LTPP IMS database that were used to sort the skid numbers into functional classes used in this project. The Multilane class includes both divided and undivided sections because information needed to separate the records was not available in the database. Approximately 90 of the 687 records (13 percent) did not have sufficient data to be placed into a functional class.

Table B-28 lists the 15th, 50th, and 85th percentile skid numbers for each functional class along with the number of records available for the class. Except for rural freeways, each functional class had less than 100 records. Figures B-22 and B-23 illustrate the cumulative frequency curves for each functional class.

SUMMARY

Several items can affect vehicle braking distances. Some of these issues, especially for heavy trucks, are significant. Using controlled rather than locked-wheel braking was a recommendation from NCHRP 270. Olson et al. argued that drivers will attempt to maintain steering ability of their vehicle especially when braking on a horizontal curve or during a lane changing maneuver. Using controlled-braking distances rather than locked-wheel braking distances results in longer stopping distances. Widespread use of antilock brake systems could resolve the issue of using controlled versus locked-wheel braking. Antilock brakes would provide the driver with steering control and allow the brakes to utilize peak braking friction.

Before antilock brakes can be assumed as the "solution" to the braking issue, additional information on the availability of the system is needed. As shown in Figure B-3, ABS systems represented only a small portion of the new vehicle passenger car sales (as of 1990). Current advertising, especially for domestic passenger cars, indicates that the percentage of new cars with ABS is higher. The data in Figure B-4 suggests that 10 or more years will be required before ABS are in more than half of the passenger car vehicle fleet. Even more years will be required before ABS represent a significant portion of the heavy vehicle fleet, even though all new trucks are required to have an antilock system by March 1, 1999.

While several studies are available that present measured braking distances, few studies provide information on driver's reactions when confronted with a hazard. There are several reactions to a hazard including the current assumption of a locked-wheel stop. Other reactions can include attempting a complete lane change, slowing and driving around the hazard, or bringing the vehicle to a stop using a controlled-braking maneuver. Drivers may also perform differently depending upon the type of braking maneuver. A driver with an antilock brake system will react differently from the driver that is having to modulate the brakes to maintain vehicle control. While several of the studies

TABLE B-27. LTPP IMS Variables Used to Determine Functional Classes.

Functional Classes	Functional Classes in the LTPP IMS		Number of Lanes	Shoulder Width
Rural Freeway	Rural Principal Arterial–Interstate	Rural Principal Arterial–Other	≥ 2	paved or unpaved
Rural Multilane	Rural Minor Arterial Rural Major Collector	Rural Minor Collector Rural Local Collector	≥ 2	paved or unpaved
Rural Two Lane High Type	Rural Minor Arterial Rural Major Collector	Rural Minor Collector Rural Local Collector	1	≥ 6 feet
Rural Two Lane Low Type	Rural Minor Arterial Rural Major Collector	Rural Minor Collector Rural Local Collector	1	< 6 feet
Urban Freeway	Urban Principal Arterial–Interstate	Urban Principal Arterial–Other	≥ 2	paved or unpaved
Urban Multilane	Urban Other Principal Arterial Urban Local	Urban Minor Arterial Urban Collector	≥ 2	paved or unpaved
Urban Two Lane High Type	Urban Other Principal Arterial Urban Local	Urban Minor Arterial Urban Collector	1	≥ 6 feet
Urban Two Lane Low Type	Urban Other Principal Arterial Urban Local	Urban Minor Arterial Urban Collector	1	< 6 feet

TABLE B-28. Skid Numbers From LTPP IMS Database.

Functional Class	Percentile			Number of Records
	15th%*	50th%	85th%	
Rural Freeway	40	48	56	426
Rural Multilane	34	52	60	20
Rural Two-Lane High	35	45	55	43
Rural Two-Lane Low	31	40	49	6
Urban Freeway	34	43	51	72
Urban Multilane	40	51	59	23
Urban Two-Lane High	40	49	54	6
Urban Two-Lane Low	46	46	46	1
All functional class records	38	47	54	597
All records (includes those records without classification information)	38	47	56	687

* *15 percent of the pavements have this skid number or worse (or, 85 percent of the pavements have this skid number or better).*

examined the difference in braking distance when ABS are present or when a driver is attempting to maintain steering control rather than locking the wheel, these studies usually had professional drivers performing the tests. Emphasis was also placed on the distance to stop rather than explaining the driver's actions. Having an appreciation of how drivers respond to different roadway objects could result in a better stopping model or could result in a more educated decision on which model is most appropriate for design.

An area where actual data is either sparse or not readily available is pavement friction. Two states and the one national database provided skid data for use in this project; however, even though the California data had over 74,000 records for two years, it is only for 40 mph. Verifying the validity of the assumed friction coefficient for other speeds would require additional data collection efforts. The extensive manipulation required to access the data is an example (and a reason) why skid number distributions are not readily available.

Using a 15th-percentile value, the Texas data indicated that all functional classes of roads in Texas have a skid number greater than or equal to the 32 value assumed in the AASHTO stopping sight distance model. The skid numbers from the SHRP database also were greater than the AASHTO assumed values except for the Rural Two-Lane Low class. It had a skid number of 31. The California pavements in the rural area were at or near the 32 value; however, except for freeways, the urban areas all had values below 32.

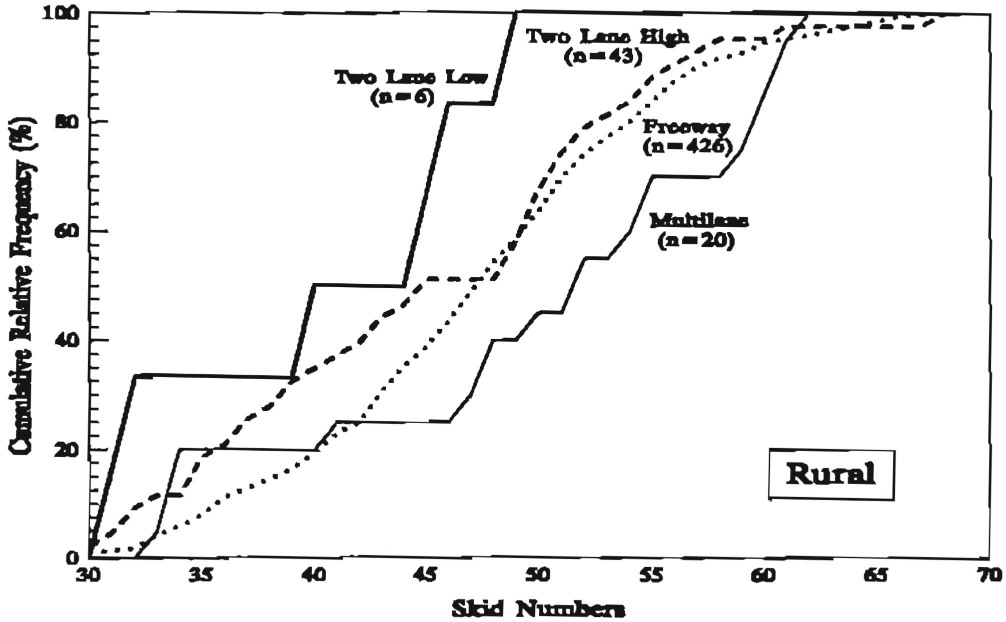

FIGURE B-22. Cumulative Skid Numbers from Rural SHRP Data.

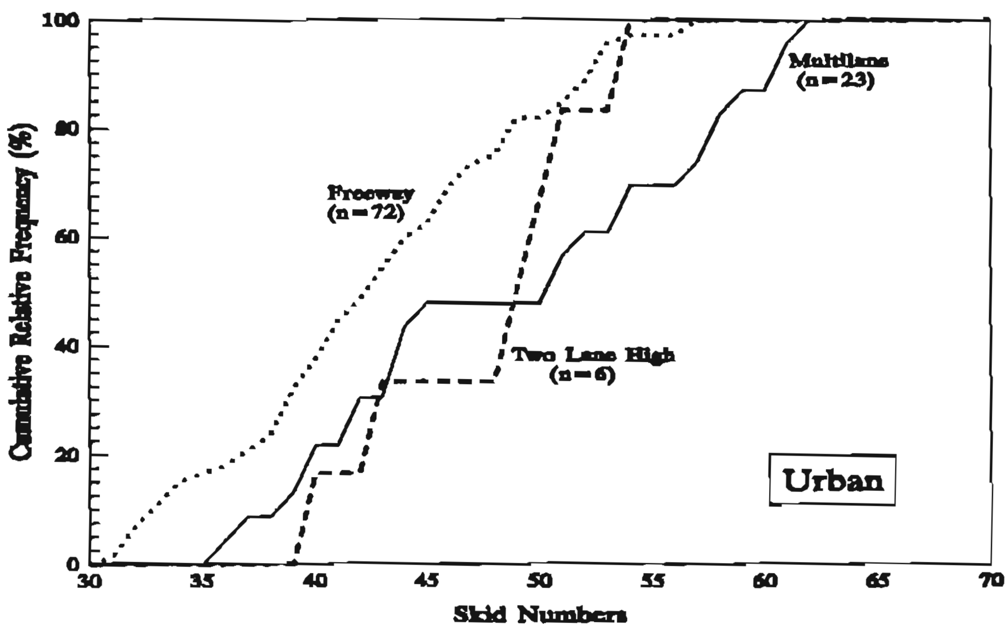

FIGURE B-23. Cumulative Skid Numbers from Urban SHRP Data.

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APPENDIX C -

DRIVER PERFORMANCE STUDIES

INTRODUCTION

As defined by the American Association of State Highways and Transportation Officials (AASHTO), three parameters control the minimum length of vertical curve: required stopping sight distance, driver eye height, and object height. This minimum length of curve provides the required stopping sight distance as a minimum along the entire length of the vertical curve. The required stopping sight distance is dependent on two fundamental components (perception-brake reaction and braking distances). Researchers should examine these components to determine their appropriateness and recommended values for geometric design of highways.

Stopping sight distance is the defined as the distance traveled during a perception-brake reaction time (PBRT) of 2.5 seconds at the design or initial speed, and the distance traveled during a locked-wheel braking maneuver from the design or initial speed. The perception-brake reaction component is based upon the assumption that "nearly all drivers under most highway conditions," traveling at or near the design speed, will require 2.5 seconds or less to perceive and react to a hazardous object that is in the roadway (*1,2*). The braking component is the distance required for this driver to perform a locked-wheel braking maneuver with worn tires on wet pavement and stop his/her vehicle before striking the object. In the AASHTO stopping sight distance model (SSD model), the assumption of a locked-wheel braking maneuver is represented as a constant deceleration from the onset of brake application to reaching a stopped position.

The objective of the driver performance studies was to evaluate driver braking characteristics to an unexpected object encountered in the roadway. These characteristics included perception-brake reaction time, driver/vehicle deceleration performance, and the overall braking distance. Researchers evaluated the effects of antilock brakes on these characteristics. The driver performance studies also focused on driver/vehicle deceleration characteristics to determine if AASHTO's locked-wheel, constant deceleration model is an adequate driver-vehicle design model. The following tasks were performed to accomplish the objectives of these studies:

- Develop a study designed to measure a driver's perception-brake reaction time, deceleration characteristics during braking, and braking distance in a controlled testing environment;
- Collect the laboratory and field data necessary for the analysis;
- Analyze the data to determine the braking characteristics; examine the effects of antilock brakes on the braking maneuver; compare and contrast these values with current AASHTO values; and make recommendations based on the findings of this study.

This appendix, *Driver Performance Studies*, is divided into six sections. Section 1, *Introduction*, includes the introduction, objectives, and organization of the appendix. Section 2, *Literature Review*, includes a review of previous braking performance studies. Section 3, *Methodology*, describes the study design, including the procedure for collecting and analyzing the driver braking performance data. Sections 4 and 5, *Results*, discuss the findings of the driver braking performance studies. The conclusions and recommendations from these studies are presented in Section 6, *Conclusions and Recommendations*.

LITERATURE REVIEW

One of the most important requirements in highway design is the provision of adequate stopping sight distance at every point along the roadway. Horizontal and vertical curves can limit available sight distance; however, when designed according to AASHTO criteria, available stopping sight distance at each and every point along the curve is greater than or equal to the required stopping sight distance. The design of vertical and horizontal curves is dependent on the required stopping sight distance, as a minimum, at each point on the roadway. This section reviews perception-brake reaction time and the deceleration studies.

Perception-Response Time Component

Perception-reaction time, or more appropriately, perception-brake reaction time, is the summation of perception time and the time for movement of the driver's foot from rest to the brake pedal, or brake reaction time. Brake reaction time was assumed as one second in 1940 (*3*); and, since that time, no changes have been made in the recommendation for this value. Total perception-brake reaction time in 1940, however, ranged from two to three seconds, depending upon design speed. In 1954, the *Blue Book* (*4*) adopted a policy for a total perception-response time of 2.5 seconds for all design speeds. The *Blue Book* (*4*) stated "available references do not justify distinction over the range in design speed." Since *available references* were not cited, the reason for this change is not clear; however, no *known references* have been found in the literature that support this distinction.

In addition to state and national geometric design policies, researchers over the last 25 years have conducted several well-documented studies to quantify driver's perception-brake reaction times. Some studies have supported the 2.5 second standard, while other studies have supported other values. These studies are discussed in the following sections.

Perception-Reaction Time Models. Assuming that only minor modifications to the driver-related parameter of the basic SSD model are necessary, the starting point for improving the model should be a thorough examination of the rationale and basis for perception-brake reaction time models. Ever since the Dutch physiologist Donders began speculating about central processes involved in choice and recognition reaction times in the mid-nineteenth century, researchers have developed many models using this process. The basis for some models used empirical work, while others used the theoretical study of inferred processes with laboratory verification. Most of the models have involved some aspect of chaining (i.e., a series of successive elements beginning with stimulus impact upon a sensory receptor and ending with initiation of a motor response).

In the early 1950s, information theory took a dominant role in experimental psychology. The linear equation, now known as the Hick-Hyman "Law," expresses a relationship between the number of alternatives that need to be sorted out to decide the response and the total reaction time. The following equation can express this relationship:

$$RT = a + bH_s \quad [1]$$

where: RT = total reaction time;
a = intercept of basic process latency;
b = increment of added process time resulting from each added bit of stimulus information; and
H_s = information (in bits) in stimulus, $\log_2(1/p)$, where p is the probability of each of m equally probable alternatives.

Underlying this formulation is the two component concept: part of the total response is common to all reactions, and part of the total response depends on choice variables. The latter part varies with such factors as sensory modality, intensity of stimulus, degree of expectancy, and other factors. An early model of this aspect was reported in "Vision in Military Aviation" (*5*). This serial chaining model is directly applicable to the stopping sight distance situation.

In Wulfek's 1953 model (*5*), detection and the subsequent elements of the perception process are differentiated from one another as shown in Table C-1. Besides the individual event descriptions, the tabular values include estimated values for the event duration and the cumulative time from initial detection. The model then switches to an adaptation of the Hick-Hyman "Law" to arrive at an estimate of total perception-brake reaction time. For example, applying the information in Table C-1 to the driving situation, Wulfek's model predicts that a motorist traveling at 60 mph will travel 90 feet during the detection-recognition process (88 ft/sec * 1.03 sec).

By choosing an appropriate foot movement model to add to this detection-recognition model, researchers have been able to use the resultant composite models to predict the total perception-brake reaction time. Foot movement from the accelerator to the brake pedal can be described by one of several variants of Fitt's Law. Brackett reported a mean foot-movement time of 0.31 seconds, with a standard deviation of 0.11 seconds over several different brake pedal configurations (*6*). Using the 85th percentile extrapolation from these data, 0.47 seconds (0.31+0.16) would be a good estimate for an 85th percentile foot movement time. Thus, an estimate of most driver's perception-brake reaction time would be 1.5 seconds (1.03+0.47) or less.

A recent model proposed by Hooper and McGee (*7*) has a breakdown of perception-brake reaction time for each component of the process with estimates that represent the 85th percentile driver. Table C-2 shows McGee's estimated values for the individual elements of response process, and indicates a total PBRT of 3.2 seconds. Because it is unlikely that a single driver will produce 85th percentile value for each of the individual elements, 3.2 seconds probably represents the upper limit for an 85th percentile driver's PBRT.

McGee and Hooper further state that if a panic maneuver is necessary, it is likely that decision time is instantaneous with the moment of perception (*7*). The 85th percentile PBRT, without a decision component, would be 2.35 seconds (3.2-0.85). These examples would predict 85th perception-brake reaction time for stopping sight distance situations to be between 1.50 and 2.35 seconds, well below the 2.5 second value assumed by AASHTO.

In recent years, models to predict human perception-brake reaction time have become probabilistic, starting with Fitts and his *random walk* model, to more recent stochastic network models such as those proposed by Wickens (*8*). Wickens discusses the assumption of linearity in the Hick-Hyman "Law," and suggests that reaction time tends to follow an exponential function if the decision is in the context of danger or severe penalty. In other words, the larger the amount of information processed (i.e., the nature of the decision and the number of alternatives the driver has to choose from in that situation), the longer it takes to react. At some point, the reaction time curve becomes asymptotic to the Y-axis and the driver does not react at all.

Clearly, perception-brake reaction time is not always a simple reaction time, but represents the total time it takes a driver to detect an object, recognize it as a hazard, decide upon an action, and initiate that action. The more complex the decision, the longer the response time. Fortunately the decision in a stopping sight distance situation is relatively simple. In the AASHTO model, researchers presume the

TABLE C-1. Detection Components in the PRT Process (5).

Event	Event Duration (Seconds)	Cumulative Time (Seconds)
Sensation	0.10	0.01
Eyes start to slew	0.18	0.28
Eye movement	0.03	0.31
Accommodation/adjust	0.07	0.38
Recognition	0.65	1.03

TABLE C-2. Elements of Perception-Brake Reaction Time (7).

Element	Time (Seconds)	Cumulative Time (Seconds)
Perception		
Latency	0.31	0.31
Eye movement	0.09	0.40
Fixation	0.20	0.60
Recognition	0.50	1.10
Decision	0.85	1.95
Brake Reaction	1.24	3.19

driver's response consists of moving the foot from the accelerator to the brake to initiate braking. They further assume that required stopping sight distance is determined when the driver immediately applies the brake pedal with sufficient force to lock the wheels.

From this point in time, the model considers no further driver response. The driver is removed as a controller of the vehicle and the laws of physics as they relate to speed, tire-pavement friction, and roadway grade control. Values used to represent these latter variables differ as a function of design speed, pavement type and condition, and roadway alignment. Perception-brake reaction time is assumed constant for all combinations of conditions.

As previously noted, the AASHTO model uses a 2.5 second perception-brake reaction time for all stopping sight distance calculations. A model more sensitive to actual human behavior would require this parameter to vary as a function of both vehicle speed and highway type; however, although such variation seems reasonable, no evidence exists in the literature to support this distinction. One proponent of this concept has hypothesized stopping sight distance perception-brake reaction times for different types of roadways as follows (*9*):

- Low volume roadways 1.5 seconds
- Two-lane primary rural roadways 3.0 seconds
- Multilane urban arterial roadways 2.5 seconds
- Rural freeway 2.5 seconds
- Urban freeway 3.0 seconds

McGee (*10*) recommended different decision sight distance perception-response times for different design speeds. In both studies, the range of recommended values is from 1.5 to 3.0 seconds, which encompasses AASHTO's 2.5 second value. One interpretation of these recommendations is that 2.5 seconds is inclusive of nearly all drivers under nearly all stopping sight distance situations, and it is only in complex decision situations, such as intersections or interchanges, that longer perception-response times are needed.

Perception-Brake Reaction Time Studies. As defined by the AASHTO stopping sight distance model, drivers respond to unexpected or emergency situations by immediately initiating locked-wheel braking. The 2.5 second perception-brake reaction time specified in the model probably overestimates perception-brake reaction time for all but the most seriously impaired drivers. Implicitly the model's very conservative perception-brake reaction time probably accounts for a number of other driver-related variables, including any differences due to roadway functional classification, mentioned in the preceding discussion.

Gazis, Herman, and Maradudin (1960). One of the earliest studies to investigate perception-brake reaction times of unalerted drivers was conducted by Gazis in the late 1950s (*11*). Recognizing the limitations of studies conducted in a testing environment where drivers are unduly alert, this study took a different approach than the laboratory-type tests. The study measured the time difference between the onset of the yellow of a traffic signal and the appearance of brake lights on approaching vehicles. Response times at six different locations ranged from 0.6 to 2.4 seconds, with an average of 1.14 seconds. Although this value may seem lower than expected for a truly *surprised* condition, it should be noted that the study was conducted in an urban environment where drivers are more alert and aware of surrounding traffic conditions.

Johansson and Rumar (1971). One of least documented but most highly referenced studies on driver-brake reaction times was conducted by two Swedish researchers at the University of Uppsala in 1971 (*12*). The primary importance of this study is the fact that the 1990 and 1984 AASHTO *Green Books* (*1,2*) identify this study as the fundamental basis for the 2.5 second perception-brake reaction time used in the stopping sight distance equation. AASHTO states that "for approximately 90 percent of the drivers (in the Johansson and Rumar study), a reaction time of 2.5 seconds was found to be adequate."

Johansson and Rumar stopped 321 drivers about to travel a rural highway in Sweden (*12*). These drivers were told to listen for a loud horn to sound at some point along a 10 km stretch of roadway. When they heard the sound, they were to touch their brakes, and then drive on. A researcher at the location where the horn was sounded concealed himself in such a way that he could see the brake lights of the passing vehicle. This individual also controlled the horn, which also started a time clock. When the researcher saw the brake lights come on, he stopped the clock. The researcher's simple reaction time to brake onset also was measured in a separate *calibration* study to correct for the lag otherwise inherent in such an approach. Thus, the 321 drivers represented an alerted, but otherwise, naive driver population.

In a separate study, five drivers agreed to allow a timer and clock circuit to be installed in their cars. The timer was set for time intervals up to one hour so that it would begin or resume its timeout each time the ignition was turned on, and stopped when the vehicle was switched off. Sometimes a week or more would elapse before the buzzer sounded if a driver performed several short trips. At the sound of the buzzer, the driver lightly touched the brake pedal to shut off the buzzer. This action stopped a clock/recorder started by the buzzer. To assure reasonably reliable response times to the buzzer, the first three data points for each of the drivers were dropped. This approach was a *surprise* condition for the driver because so much time had elapsed since the last time the buzzer came on. These drivers were also run in the same fashion as the 321 drivers described above to obtain a baseline *anticipated* reaction time. Table C-3 presents the *surprise* and *anticipated* findings for these five subjects (*12*).

To compare the differences between a *surprise* perception-brake reaction time and an *anticipated* perception-brake reaction time, Johansson and Rumar calculated an empirical correction factor, or the factor of difference between a *surprise* reaction time and an *anticipated* reaction time (*12*). The mean *surprise* PBRT divided by the mean *anticipation* PBRT gives a correction factor of 1.35:

$$\textit{Correction Factor} = \frac{\textit{Surprise PBRT}}{\textit{Anticipated PBRT}} = \frac{0.73\ Sec}{0.54\ Sec} = 1.35 \qquad [2]$$

Johansson and Rumar suggest that perception-brake reaction times collected under the usual conditions in which the driver is anticipating that he or she will need to respond can be used for estimating a *surprise* PBRT by simply multiplying the *anticipated* PBRT by 1.35 correction factor. This factor could be used to adjust *anticipated* perception-brake reaction times from other studies to produce a larger database of *surprise* perception-brake reaction times.

Sivak, Olson, and Farmer (1982). Sivak (*13*) took a different approach to measuring perception-brake reaction times: a *covert* or unsuspecting approach. A *convoy* approach was used in which a lead car, equipped with a special circuit to onset the brake lamps without actually braking the vehicle, was maneuvered in front of an unsuspecting driver in a traffic stream on a roadway. A car following behind the test *subject* was equipped with a Doppler radar and a recording system. When it was safe to do so, the lead car driver flashed the stop lamps at the test *subject*. At the same time, a radio signal was sent to the following car to record the time at which the test *subject* took his foot off the accelerator pedal and/or braked.

TABLE C-3. Summary of Perception-Brake Reaction Times for Five Subjects (*12*).

Situation	Observations		Median (seconds)		Range (seconds)	
	Surprise	Anticipated	Surprise	Anticipated	Surprise	Anticipated
Subject						
A	10	10	0.85	0.60	0.7-1.1	0.5-0.7
B	10	10	0.60	0.50	0.6-1.0	0.5-0.8
C	10	10	0.90	0.55	0.7-1.0	0.5-0.8
D	10	10	0.70	0.55	0.6-0.7	0.5-0.8
E	10	10	0.60	0.50	0.5-0.9	0.4-0.6
Average			0.73	0.54	0.5-1.1	0.4-0.8

The headway between vehicles was set at approximately one second. This situation might be considered representative of the stopping sight distance situation, since the unsuspecting driver was not prepared for a sudden braking event in front of him or her in what presumably was free-flowing traffic. The situation is similar to suddenly encountering an obstacle or hazard which requires an immediate braking response, a simple decision. Sivak et al. reported a mean perception-brake reaction time of 1.2 seconds (*13*).

Wortman and Matthias (1983). A similar approach to Gazis, but on a much larger scale, was conducted by Wortman and Matthias (*14*). The researchers measured *covert* perception-brake reaction times to the onset of the yellow phase of traffic signals at six intersections in Phoenix and Tucson, Arizona. Perception-brake reaction times were derived from timing the interval after a traffic signal changed to amber to the onset of approaching vehicle's brake lights. This study reported PBRT values ranging from 0.5 to 2.1 seconds, with a mean of 1.3 seconds.

It should be noted that perception-brake reaction times to traffic signals represent a traffic situation different than encountering an unexpected situation or object in the roadway. The driver was presumably aware that a signal was ahead and there was a possibility that drivers anticipated the onset of the yellow phase; however, any benefit gained from anticipation was probably offset by the time required to decide between stopping or proceeding through the intersection.

Chang, Messer, and Santiago (1984). In a study similar to those of Gazis and Wortman, Chang (*15*) reported a mean perception-brake reaction time to signal changes at intersections of 1.3 seconds. Again, it should be noted that benefits gained from anticipating the signal change interval were probably partially offset by the decision to stop or proceed through the intersetion. Because of the decision that must be made, the data probably represents similar perception-brake reaction times to what drivers might exhibit to a unexpected object in the roadway.

NCHRP 270, Olson et al. (1984). This study was performed for NCHRP 270 to determine estimates of perception-brake reaction time (*16*). Olson also conducted a *surprise* study and an *alerted* study to evaluate driver perception-brake reaction times as a function of several variables. Olson's studies were conducted on a rural, two-lane roadway using ten volunteer drivers, five of which were 18 to 40 years in age and five of which were 60 to 84 years in age. The object used for both studies was a relatively low-contrast 6-inch piece of foam rubber. It was located just beyond the crest of a vertical curve for the *surprise* study, but was moved to different locations beyond the crest for the *alerted* study.

In the *surprise* study, each driver reacted (usually by steering out of the way) to the unexpected object that suddenly appeared over the crest of the vertical curve. After this initial maneuver, five more trials were conducted under anticipated conditions, where each driver was asked to approach the vertical curve and to release the accelerator and tap the brake as soon as they saw the obstacle. A calibration test run was conducted prior to the *alerted* study to determine at what point the object could be detected by the driver. The calibration allowed the translation of the data to perception-reaction time values (*16*).

A final *alerted* test was conducted at the completion of the first two studies. A 4-inch diameter lamp was attached to the hood of the vehicle, and at some point on the return trip from the rural site, the lamp was activated. The driver was asked to release the accelerator and to tap the brake as soon as they saw the lamp illuminate. This test was repeated five times for each driver and referred to as the *brake* trials. Table C-4 summarizes the findings of Olson's three studies.

The *surprise* and *alerted* perception-brake reaction time values were essentially the same for both the younger and older drivers in Olson's studies. A correction factor, similarly calculated previously by Johansson and Rumar (*12*), was not calculated by Olson, but could be inferred from the data presented in Table C-4. The mean *surprise* perception-brake reaction times divided by the mean *alerted* perception-brake reaction times for both age groups yields a correction factor of 1.48: .

$$\textit{Correction Factor} = \frac{\textit{Surprise PBRT}}{\textit{Alerted PBRT}} = \frac{1.10 + 1.05\ Sec}{0.70 + 0.75\ Sec} = 1.48 \qquad [3]$$

This correction factor is slightly higher than what Johansson and Rumar calculated, but indicates the general tendency of the correction factor being greater than 1.0 but less than 2.0. The rudimentary procedure for calculating the perception-brake reaction values in the Olson study may have introduced error; and again, a more complex scenario might cause this factor to be slightly larger than presented here.

The previously described perception-brake reaction time studies and several others from the literature are summarized in Table C-5. The *remarks* column in Table C-5 provides very brief descriptions of how each study was conducted. Note that three studies under the *covert* category involve drivers who were not aware that they were test subjects. The consistency and large samples sizes associated with the *covert* studies of Sivak, Wortman, Chang, and their various collaborators suggest that these findings should be considered a *good estimate* of the true perception-brake reaction time in response to a signal change interval.

TABLE C-4. Summary of Perception-Brake Reaction Times for Ten Subjects (*16*).

Condition	Percentile	Perception-Brake Reaction Time (seconds)	
		Younger Drivers	Older Drivers
Brake	50th	0.60	0.65
	85th	0.70	0.75
	95th	0.82	1.10
Alerted	50th	0.70	0.75
	85th	0.92	1.00
	95th	1.15	1.28
Surprise	50th	1.10	1.05
	85th	1.32	1.40
	95th	1.60	1.60

The study by Lerner (*17*) in the *surprise* category is significant because he compared perception-brake reaction times for older drivers to those of younger drivers in a stopping sight distance situation. Fifty-six of the 116 subjects reacted to an unexpected object by braking; the remainder reacted by steering and/or braking. The mean PBRT was 1.5 seconds with a standard deviation of 0.4 seconds. The 85th percentile PBRT was 1.9 seconds, and the longest observed PBRT was 2.54 seconds. Importantly, **there was no significant difference in mean perception-brake reaction time due to age**. Although important for stopping sight distance situations, this result should not be interpreted to suggest that there is no age-related slowing for more complex driving situations (*17*).

The Johansson and Rumar data (*12*) and the other two studies (actually one study conducted by Olson, but divided by subject age) summarized in Table C-5 under behind-the-wheel *alerted* driver conditions are also quite consistent with one another. They should be considered *good estimates* of the true perception-brake reaction for this situation. Note that as might be expected, perception-brake reaction times for unexpected signals and objects were longer than perception-brake reaction times for expected signals and objects.

Mean and upper percentile estimates were provided for the different types of studies summarized in Table C-5. These calculations were done to arrive at a *composite* estimate of perception-brake reaction time. The results of this estimation procedure are summarized in Table C-6. If a ratio is calculated between the *surprise* or *covert* driver perception-brake reaction times and either *simulator* or *behind the wheel* perception-brake reaction times, a correction factor can be computed to estimate the PBRT in a driving situation from experimental data. The correction factors across percentiles are provided in Table C-7, together with a ratio between *covert* versus *surprise*. The *covert/surprise* ratio increases as the extremes of the distributions are reached, from 1.25 to 2.00 as the percentiles increase from 85th to 99th.

Tables C-5 to C-7 suggest large differences between the covert data and the data collected in various ways in an experimental approach for measuring perception-brake reaction time. The most *realistic* condition, drivers induced to make stopping decisions by faking braking ahead or responding to a traffic signal, yields PBRT estimates which makes the AASHTO constant of 2.5 seconds seem conservative, since the 95th percentile from the combined *covert* data is 2.5 seconds. Any of the *experimental surprise* conditions, whether for young or old drivers, only reach 1.75 seconds at the extreme 99th percentile.

Perception-Brake Reaction Times of Impaired Drivers

Drugs and Alcohol. It is generally accepted that drivers are not always at their peak performance when driving. They may be impaired by drugs, legal or otherwise, they may be ill, and they may be sleepy. Drugs such as alcohol, cannabis, and barbiturates tend to increase both simple (single stimulus, single choice) and complex (multiple stimulus, multiple choice) reaction time (*20*). Test data shows a 0.015 percent increase in reaction time for each 1.0 percent increase in blood alcohol (*21*). Some antihistamines can have a similar effect. Minor tranquilizers may not affect simple reaction time, but may affect complex perception reaction time if the user is not habituated. On the other hand, stimulants such as amphetamine and cocaine tend to shorten both types of reaction time. The tendency of users to increase their dosage levels in an addiction process may lead to a much impaired reaction time if they abruptly interrupt drug usage. This behavior is referred to as the *crash* phenomenon (*20*). A study of the effects of drugs on driving (*22*) provides consistent findings. Secobarbital, marijuana, diapezam, and alcohol all adversely affected emergency stop and turnoff maneuvers, as well as tasks requiring detection abilities.

TABLE C-5. Summary of Surprise and Alerted Perception-Brake Reaction Time Studies.

Study Condition: Surprise (Unspecting Driver), Perception to Start of Brake Actuation					
	N	Ages	Mean	Std.Dev.	Remarks
Covert:					
Sivak *et al.* (*13*)	1644	Mix	1.21	0.63	Traffic Signal
Wortman (*14*)	839	Mix	1.30	0.60	Traffic Signal
Chang *et al.* (*15*)	579	Mix	1.30	0.74	Traffic Signal
MEAN ESTIMATES			1.27	0.66	
Surprise:					
Olson, Sivak (*16*)	49	Young	1.10	0.15	Unexpected Object
Olson, Sivak (*16*)	15	Old	1.06	0.10	Unexpected Object
Lerner (*17*)	56	Mix	1.50	0.40	Unexpected Object
MEAN ESTIMATES			1.28	0.20	
Study Condition: Anticipated (Alerted Driver), Onset to Start of Brake Actuation					
Simulator:					
Bracket *et al.* (*6*)	114	Mix	0.31	0.11	Onset Red Light
Retchin *et al.* (*18*)	61	Old	0.66	0.66	Bumpa-Tel Test
Retchin *et al.* (*18*)	38	Old	0.84	0.10	Bumpa-Tel Test
Cation *et al.* (*19*)	104	Mix	0.43	0.10	Onset Red Light
MEAN ESTIMATES			0.56	0.10	
Behind the Wheel:					
Olson, Sivak (*16*)	49	Young	0.72	0.11	Expected Object
Olson, Sivak (*16*)	15	Old	0.73	0.10	Expected Object
Johansson, Rumar (*12*)	321	Mix	0.75	0.28	Expected Horn
MEAN ESTIMATES			0.73	0.16	

TABLE C-6. Mean and Percentile Estimates of Perception-Brake Reaction Times.

Condition	50th (second)	85th (second)	95th (second)	99th (second)
Covert Observation	1.27	1.87	2.52	3.51
Experimental Surprise	1.28	1.49	1.61	1.75
Simulator	0.56	0.66	0.75	0.85
Behind the Wheel	0.73	0.90	1.03	1.21

TABLE C-7. Mean and Percentile Correction Factors.

Ratio	50th (second)	85th (second)	95th (second)	99th (second)
Surprise/Simulator	2.29	2.26	2.15	2.06
Surprise/Behind the Wheel	1.75	1.66	1.56	1.45
Covert/Simulator	2.27	2.83	3.35	4.12
Covert/Behind the Wheel	1.73	2.08	2.44	2.89
Covert/Surprise	0.99	1.25	1.56	2.00

Medical Conditions. Drivers who are either acutely or chronically ill may exhibit a lengthened simple or complex reaction time or both, if the illness results in fatigue. Conditions such as diabetes mellitus can render drivers relatively incapacitated rather suddenly. Other illnesses, such as multiple sclerosis and various kinds of arthritis can also impair a person's reaction time. Prediction of the effects of disease and medical disability on simple and complex reactions time can be difficult, according to the Merck Manual (*23*). A tentative conclusion may be that illnesses that still permit a driver to drive have a relatively minor effect on perception-response time.

Fatigue. In a recent study by Tilley and Bohle (*24*), high school youths were tested for simple reaction times under sleep deprivation and two types of activity. One group danced all night, the other simply amused themselves in a common room. Mean simple reaction times increased for both groups from about 0.23 seconds in the before case to between 0.25 and 0.27 seconds after 8 hours of sleep deprivation. Those who danced all night had a shorter mean reaction time after sleep deprivation than those who did not. Another study of simple reaction time performance under conditions of both sleep deprivation and impairment by alcohol found that both conditions, alone and in combination, lead to lengthened perception-response times (*25*). On the other hand, simple reaction times taken of young Swedish army volunteers in a driving task during various points in their diurnal cycle showed only very minor effects attributable to circadian rhythms alone (*26*).

Other Aspects of Perception-Brake Reaction Time Modeling

The description of the often-mentioned "design driver" should also include reasonably adequate perception-response time models that can predict not only what drivers **can** do (competency), but also what they can be **expected** to do (performance). Lerner (*27*) pointed out that in such modeling, there are two perhaps complimentary approaches. The classic approach is to set up a wide variety of (simulated) highway situations and measure perception-response time. The model is then expressed as a transfer function in terms of input and output. The other approach involves laboratory measures of component processes, such as the model proposed by Hooper and McGee (*7*). This latter model arises from a summation of times accruing to the various components of the perception-reaction process, such as eye movement, target acquisition, decision making, etc.

Two often overlooked considerations in this latter approach should be noted. First, analogous to anthropometric measures, the distribution of times is different for each of the pertinent components. For example, an individual who exhibits 85th percentile target acquisition time cannot be assumed to be in the 85th percentile of decision making times. Stated differently, it is highly unlikely that the sum of the 85th percentile times for each component of perception-brake reaction time will provide an adequate estimate of the 85th percentile total PBRT. A second important consideration is the rather unrealistic assumption that all of the components are strictly sequential.

Braking Performance Studies

Discrepancies in assumed driver braking performance arise amongst some of the available literature. Some confusion exists from the pre-antilock brake era (1960s) and the more recent studies. A study conducted by Starks and Lister in 1955 states that in an emergency situation "it is suspected that drivers apply their brakes as hard as possible" (*28*). This idea contrasts to the more recent 1984 NCHRP Report 270 in which the authors state that drivers will *modulate* their braking so as to maintain directional control (*16*).

Limited research has been conducted on driver performance and deceleration characteristics in emergency stopping situations; however, extensive research has been conducted on braking performance of trucks and other vehicles, including a National Highway Traffic Administration (NHTSA) study to evaluate antilock braking performance of two-axle trucks (*29*), and a light truck and passenger car braking performance study in the mid-1980s (*30*). These research activities have focused primarily on vehicle performance characteristics, such as in extreme vehicle-maneuver conditions (i.e., maximum braking).

Braking Distance From Deceleration Rates, Shadle et al. (1983). In a 1983 SAE paper, Shadle, Emery, and Brewer reported on braking tests on ten 1980 or later vintage passenger cars, three pickup trucks, and one van (*31*). The vehicles were instrumented to provide deceleration histories, pedal forces, and other parameters. The vehicles were not actually braked to a hard stop, but rather after wheel lock was obtained, measurements of brake pedal force were made. Stopping times were extrapolated using standard equations of kinematics and deceleration values determined from the brake pedal force. Vehicle braking distances were measured when both loaded and with only the driver aboard. Readings were taken just before either the front or rear axle wheels locked and then with either or both axles locked. Highway surface conditions were characterized as either *good* or *poor*, and either *dry* or *wet*. Table C-8 lists the ranges of braking distances and the mean values calculated from the deceleration values presented in the paper for vehicles with one axle locked. The data generated cannot be considered completely empirical, since partial stops were extrapolated.

Deceleration for a Signal Change Interval. A study by Chang et al. (*15*) on perception-brake reaction times mentioned previously also measured dry-pavement deceleration characteristics to traffic signal change intervals. They found that the mean decelerations for the two intersection approaches studied were 10.5 and 12.5 ft/sec^2 (0.33 to 0.39 g). They also commented that the braking rate appears to be affected by approach speed, distance at yellow onset, and perception and brake reaction time (*15*). In a separate study by Wortman and Matthias (*14*), dry-pavement mean deceleration measured at six study sites was 7.0 to 13.9 ft/sec^2. The mean value for all observations from the six intersections was 11.6 ft/sec^2, a result that is consistent with Chang's findings.

TABLE C-8. Maximum Deceleration and Braking Distances for Vehicles With One Axle Locked (*29*).

Load	Pavement Quality	Maximum Deceleration (g)		Braking Distance* (feet)	
		Range	Median	Range	Median
Passenger Cars (10 post-1980 cars)					
Unloaded	Poor, wet	0.388-0.521	0.414	195-262	247
Unloaded	Good,dry	0.804-0.964	0.881	105-126	116
Loaded	Poor, wet	0.327-0.440	0.400	231-310	256
Loaded	Good, dry	0.671-0.946	0.822	107-151	125
Pickups (3 pickups, 1 representative of pickups produced in the 1970s, 2 representative of more recent models that have similar load capacity)					
Unloaded	Poor, wet	0.493-0.411	0.452	206-247	226
Unloaded	Good, dry	0.943-0.872	0.908	108-116	112
Loaded	Poor, wet	0.445-0.407	0.426	228-249	239
Loaded	Good, dry	0.924	0.924	110	110
Van (1 van, representative of vans produced in the 1970s)					
Unloaded	Poor, wet	—	0.508	—	200
Unloaded	Good, dry	—	0.953	—	107
Loaded	Poor, wet	—	0.462	—	220
Loaded	Good, dry	—	0.890	—	114

* *Calculated using the given deceleration and V = 55 mph in the following formula:* $SD = 1.47V^2/2sg$.

TABLE C-9. Deceleration Based on Figure II-17 in the *Green Book* (*1*).

Type of Deceleration	Speed (mph)	Stopping Distance (feet)	Friction	Deceleration (ft/sec^2)
Comfortable rate of deceleration	60	475	0.25	8.13
	30	180	0.17	5.37
Minimum braking distance	60	210	0.57	18.4
Dry pavement	30	50	0.60	19.3
Minimum braking distance	60	295	0.41	13.1
Wet pavement	30	70	0.43	13.8

METHODOLOGY

Because many drivers usually choose to maintain control of their vehicle rather than *stomping* the brakes in a emergency situation, the assumption of locked-wheel braking warrants a closer investigation. A more realistic driver/vehicle performance model that takes into account a controlled type of maneuver would better represent real-world conditions. Therefore, the *Driver Performance Studies* quantified driver and vehicle performance in reaction to an unexpected object in the roadway. The test conditions for drivers included not only an unexpected, or *surprise*, object situation, but also an expected, or *anticipated*, object situation taking into account the differences between the two conditions.

This research incorporated four different, but similar, field studies to determine the different aspects of driver braking performance. Different drivers were used in each of the four studies. The experimental study design measured driver performance, such as perception-brake reaction times, braking distances, and deceleration characteristics. The study design took into account vehicle handling differences and driver capabilities associated with antilock braking systems (ABS), wet and dry pavement conditions, and the effects of roadway geometry.

The first study, on a closed course, was the largest, requiring over 2,000 braking maneuvers. The next two studies, each on a closed course, were a product of the results from the first study, testing more specific and significantly different test conditions. Part A of Studies 2 and 3 evaluated driver and vehicle performance in reaction to an unexpected object scenario, and Part B evaluated performance in reaction to an expected object scenario. Study 2 also included a Part C, testing a driver's baseline perception-brake reaction time time, and a Part D, testing a control group of driver's baseline PBRT. The fourth field study, an open-road study, only measured performance data for an unexpected object scenario. Each field study is briefly described in Table C-10.

The test subjects chosen to participate in these studies included older drivers, and younger, less experienced drivers. The older driver is the fastest growing segment of the driving public and performs poorest in those factors directly relevant to stopping sight distance (i.e., visual perception, decision making lag, reaction time, and control movement time). At the other end of the physical skills and driving experience spectrum is the younger driver. Their physical skills are much better, but their driving experience is limited. It is possible, because of this lack of experience, that their decision making and braking performance may be poorer when confronted with a critical driving situation.

To cover the two extremes of the driving population, drivers less than the age of 25 and over the age of 55 were recruited to participate in these studies. Drivers from both age groups were from the local area and many were on an active participant list that is maintained for other empirical studies. Also included in the older age group were active senior members of a local church. None of the test subjects participated in more than one study.

Study 1—Closed-Course Braking Study: TTI Drivers, TTI Vehicles

Background. The purpose of this first braking performance study was twofold. First, the study served as a *pilot*study to determine the amount of testing and under what conditions the remaining field studies would be conducted. Nine TTI employees participated as subjects for Study 1. Several testing conditions, as well as several repetitions of these conditions, were established for each of the subjects. The research team hoped that by having a sizable database to analyze at the conclusion of Study 1, they could eliminate non-significant test conditions and determine necessary conditions for Studies 2 and 3. Secondly, this study provided high speed braking performance data. Previously established studies have been limited because of the dangers involved with testing at high speeds, in excess of 60 mph. This study, however, could utilize a closed-course test track and high performance drivers to obtain high speed performance data.

Test Subjects. Study 1 required extensive time, from the research team and the test subjects because of the large number of braking maneuvers performed. Each driver was required to spend several days conducting braking maneuvers, but because Study 1 served as a *pilot* study in determining the test conditions for the remaining studies, nine TTI employees were commissioned as drivers for the study. Three of the subjects were *experts*, with extensive experience in high performance driving and training. They were between the ages of 30 and 45. The remaining six subjects, three in the younger age group and three in the older age group, served as *average* drivers, more representative of the driving population.

Test Conditions. To evaluate a variety of variables that affect driver/vehicle braking performance, and to eliminate non-significant variables for future testing, several combinations of test conditions were established for the subjects in Study 1. All nine subjects performed braking maneuvers at speeds of 40 and 55 mph, and the three *expert* subjects were further tested at 70 mph. They were also tested with several other conditions: with antilock brakes enabled or disabled, with pavement conditions wet or dry, and on three geometric conditions, including a tangent section and a left and right horizontal curve. Further testing conditions included either braking at the onset of a counted-down signal (*anticipated stop*) or at a randomly selected signal onset (*surprise stop*). The conditions for the *average* and *expert* subjects are summarized in Table C-11.

Course Layout. Study 1 was conducted at the Texas A&M University Riverside Campus on three different closed-course tracks. Each course was designed for one of the three test speeds of 40, 55, or 70 mph, and all consisted of tangent and horizontal curve sections. Each of the horizontal curves were conservatively designed to maximize all available friction for the braking maneuvers. The minimum radii used for the curve designs were based upon maximum allowable

TABLE C-10. Summary of Driver Braking Performance Studies.

Study	Part	Test Condition	Test Vehicle	Test Subjects	Encounter
Study 1		Closed	TTI Vehicle	TTI	Anticipated/Surprise
Study 2	Part A	Closed	TTI Vehicle	Pool/Church	Unexpected
	Part B	Closed	TTI Vehicle	Pool/Church	Expected
	Part C	Closed	TTI Vehicle	Pool/Church	Expected
		Closed	TTI Vehicle	Church	Expected
Study 3	Part A	Closed	Personal	Pool/Church	Unexpected
	Part B	Closed	Personal	Pool/Church	Expected
Study 4		Open-Road	Personal	Pool/Church	Unexpected

TABLE C-11. Summary of Test Conditions for Study 1.

	Number of Test Conditions Per Average Driver					
Condition	Speed *(40/55)*	ABS *(On/Off)*	Pavement *(Wet/Dry)*	Geometry *(Tangent/Curve)*	# Trials	Total
Anticipated	2	2	2	3	3	72
Surprise	2	2	2	3	5	120
No Signal	2	2	2		5	40
Total						232
	Number of Test Variables Per Expert Driver					
Condition	Speed *(40/55/70)*	ABS *(On/Off)*	Pavement *(Wet/Dry)*	Geometry *(Tangent/ Curve)*	# Trials	Total
Anticipated	3	2	2	3	3	108
Planned	3	2	2	3	5	180
No Signal	3	2	2		5	60
Total						348
Total Maneuvers =	6 *Average* Drivers x 232 Runs + 3 *Expert* Drivers x 348 Runs =					2,436

side friction factors recommended by AASHTO (*1*) for a wet pavement and no superelevation. The maximum degree of curvature was also established based on these side friction factors and minimum radii.

The minimum length of each of the curves necessary to accommodate the braking maneuvers was based upon the distance from the stopping sight distance equation. A 1.0 second perception-brake reaction time and the respective design speed and corresponding friction value recommended by AASHTO were used to establish this minimum value. The lengths were then rounded up to accommodate 25-foot segments along the centerline. The course designs are summarized in Table C-12 and the location of each of the courses at the Riverside Campus is shown in Figure C-1.

The 40 mph test course was an *S*-curve bounded by two tangents at each end. The course simulated a two-lane roadway, with 12-foot wide lanes, delineated by a yellow centerline stripe and solid white pavement edge stripes. Drivers entered on a tangent from the south, made a curve to the left, traversed an intermediate tangent section, and transitioned to a curve to the right before exiting to another tangent section at the north end of the course. The *S*-curve was approximately 1500 feet in length, including the intermediate section. The tangent sections at either end became part of the taxiway and the apron area and had more than enough length to perform braking maneuvers. The intermediate tangent section, approximately 500 feet in length, was not used for braking maneuvers.

TABLE C-12. Test Course Geometric Design for Study 1.

Design Speed (mph)	Side Friction f_s	Radius R_{min}* (feet)	Degree of Curve D_{max} (degrees)	Curve Length Computed (feet)	Curve Length Retrofitted For Design (feet)
40	0.15	715	8.04	275	500
55	0.13	1,555	3.69	490	500
70	0.10	3,275	1.75	765	775

* *Assumes no superelevation or crown.*

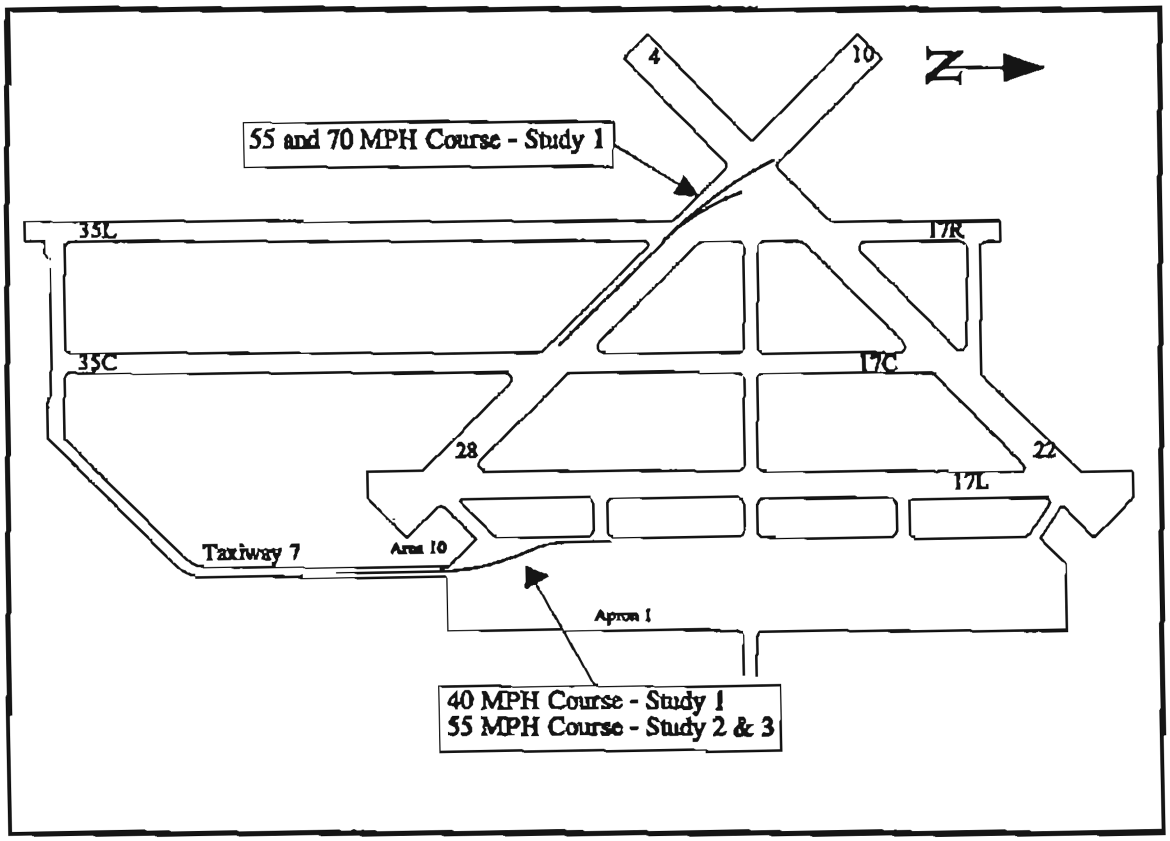

FIGURE C-1. Test Track Curve Locations for Study 1.

A small holding pond in Area 10 was used to obtain water for wet pavement braking maneuvers. An eight horsepower, gasoline driven waterpump distributed water to a hose system that was laid out at the south half of the test section. The watering system consisted of 16 water sprinklers spaced at approximately 25-foot spacing along the south tangent and along the south curve. Each sprinkler distributed a supply of approximately two gallons of water per minute, which provided a 0.05 to 0.10-inch water depth on the pavement depending upon the location. Thus, every test run involved opportunities for braking maneuvers on a tangent or a curve in either direction, and on a dry or wet pavement.

The 55 and 70 mph test courses were set up on the northwest section of Runway 28 and Runway 22. The approach tangent sections used for the braking maneuvers were the same for each course. At the point-of-curvature (PC) along this tangent, the curves diverged to the respective design radii, degree of curvature, and curve lengths. Each course was delineated by a yellow center-line pavement tape. Orange traffic cones were used to mark the pavement edges of the 12-foot wide lanes on each of the centerlines. The course could be driven in both directions, but because of the required length of curve necessary to accommodate the braking maneuvers, inadequate runway space was available to perform tangent braking in both directions. Only the approach tangent section, which could be driven from both directions, was used for this purpose.

Only one of the courses, the 55 or the 70 mph course, was driven during any one time, with the cones from the other course removed to eliminate potential driver confusion. As testing was completed on one course, the cones were removed and placed on the other course. A water truck with a gravity feed sprinkler was used to produce the wet pavement for these two courses. Because of the course layout, the conditions only allowed the course to be either wet or dry, but not both at the same time. Depending upon which course was being used, the water truck would initially make four passes on the entire course and then two passes after every two braking maneuvers. This strategy allowed either a tangent or curve braking maneuver, either wet or dry, in both directions. The water distribution, although not continuous, provided a 0.03 to 0.05 inch water depth on the pavement surface.

All three of the test course pavements were comprised of 15 to 20-foot square Portland cement concrete pavement slabs, continuously spaced, and joined by 1-inch wide expansion joints. These 12-inch thick slabs are approximately 50 years old; however, they were in relatively good condition. All test surfaces were measured at test speeds with a standard locked-wheel skid trailer, and the results are illustrated in Table C-13.

TABLE C-13. Test Course Skid Number Values for Study 1.

Location	Wet Skid Number	Dry Skid Number
40 mph Course	44.5	87.8
55 mph Course	41.2	86.3
70 mph Course	33.8	67.6*

* *Estimated based on 40 and 55 mph ratios.*

Test Vehicle and Instrumentation. A 1991 four-door Chevrolet Caprice sedan from the Texas A&M University motor pool was used for Study 1. This rear-wheel drive passenger car was regularly maintained by the Texas A&M Transportation Center and routinely checked by the research team for engine oil, brake fluid, tire pressure, tire treadwear, brake wear to the front disc brakes and rear drum brakes, and windshield wiper effectiveness. When problems were identified, vehicle repairs were immediately reported and repaired by the Center or by TTI support staff.

The vehicle was equipped with an antilock braking system. The front wheels were controlled individually and the rear wheels, on a single hydraulic circuit, were controlled together. To simulate a normal braking system, the ABS could be disabled by removing a fuse from the fuse panel, disconnecting the ABS processor. It was discovered, however, that a "dump valve" sensing brake fluid pressure in the rear wheels prevented the rear wheels from ever locking up, even under the strongest pedal pressure.

Tire treadwear and brake wear were in constant need of inspection and repair. The tires were inspected daily, maintained at a pressure of 30 psi, and records were kept to monitor the treadwear. The front tire treads usually wore quicker, and to prolong tire life, tires were rotated when the tread was below 6/32". If more than two tires had less that 4/32" tread remaining, all four tires were replaced. The front disc brake shoes and the rear brake linings and drums were inspected during tire rotation and replacement. If brake degradation affected braking performance, then the shoes and linings were repaired also. Occasional field adjustments were made to the rear drum brakes by disconnecting the ABS processor and backing up the vehicle and applying the brakes a few times.

The instrumentation in the vehicle consisted of several hardware components. A Compaq 386 laptop computer, held by the onboard test administrator in the front passenger seat, operated the data collection system. The portable computer contained a data acquisition software program, written in QuickBASIC. Linked to the laptop was an accelerometer package, located on the rear floor of the passenger compartment, providing longitudinal and lateral acceleration forces during the braking maneuvers. Also linked to the laptop through a serial connection was the *Daqbook* data box, recognized by the software and used to translate the braking data to a readable output.

Pressure-sensitive tape switches on the throttle and brake, used to monitor the placement of the driver's foot, were connected to the accelerometer package, as well as a control pendant, operated by the test administrator. The pendant contained three buttons: one for illuminating a red light-emitting device (LED) mounted on the windshield in front of the driver; one for sounding a buzzer located inside the accelerometer package; and one for placing a *mark* in the data record when necessary. This hand-held device was used to illuminate the LED to signal when the subject began the braking maneuver. The Chevrolet Caprice was also equipped with a standard *fifth-wheel* attached to the rear bumper to record longitudinal vehicle translation. A digital input into the data record allowed the research team to determine vehicle speed and braking distance for each maneuver.

The four main components, the laptop, the accelerometer package, the data box, and the fifth-wheel, were powered by an DC to AC power converter attached to the car's battery. The hardware arrangement also provided a continuous electrical circuit to the tape switches and to the control pendant.

The software program on the laptop computer required the test administrator to enter an identification code for each test subject and the beginning test run number. The administrator was then prompted to calibrate the acceleration solenoids in the accelerometer package. This prompt basically *zeroed* the translation and provided a reference for the lateral and longitudinal acceleration readings. This calibration was required each time the computer was turned on, or when the program was restarted due to a malfunction.

After this initial input, no further information was required and the program was ready for operation. The program generated an ASCII text file for each test run, which recorded time, distance, lateral and longitudinal acceleration, foot placement on the throttle or brake, and any input from the 3-pendant device. During the test run, the data was updated every 0.01 second, for a total of up to 20 seconds. A typical 10 to 15 second braking maneuver recorded by the program generated a 40,000 byte file on the hard disk. The files could then be transferred to a floppy disk and imported into other software programs for analysis.

Part of a typical data file for one test run is shown in Figure C-2. The first column is the elapsed time from the point of initiation by the test administrator in the vehicle. The timer was started 1 or 2 seconds prior to depressing the *mark* button to initiate the unexpected object, or to signaling the LED display to begin the expected braking maneuver. The second column denotes the distance in feet, relative to the point of initiation. The next five columns are binary event indicators, with "1" indicating "ON" and "0" indicating OFF". The first of these columns is the LED display indicator, or *visual* indicator. The second column is the *audio* indicator and the third column is the *mark* indicator. The fourth column is the throttle indicator, with "1" indicating pedal pressure and "0" indicating no pedal pressure. The fifth column is the brake pedal indicator, with the same binary indicators as the throttle. The last two columns indicate longitudinal (G_x) and lateral$_y$ (G) acceleration of the vehicle.

Referring to Figure C-2, the LED display was initiated at 0.66 seconds. The test subject's foot remained on the throttle until 1.08 seconds, and then the brake was depressed at 1.20 seconds. The maximum longitudinal deceleration during this braking maneuver was 0.67 g, which occurred at 1.60 seconds, and the vehicle came to a complete stop at 5.81 seconds. The program continued to run and was stopped at 7.03 seconds. Note that the distance reading did not change from 5.81 to 7.03 seconds. The *break* lines were inserted to shorten the length of this sample data file. The actual data file is 703 lines in length, or 1 line for every 0.01 second.

The reaction time was 0.42 seconds (1.08 - 0.66), the foot-movement time was 0.12 seconds (1.22 - 1.08), and the total perception-brake reaction time was 0.54 seconds (reaction time plus foot-movement time). To calculate the braking distance, the distance at the point when the brake was depressed was subtracted from the distance at the point when the vehicle came to a complete stop. In this example, the total braking distance was 178 feet (276 - 98).

Test Personnel. Two test personnel were required for the 40 mph braking studies. One person served as the test administrator in the vehicle and the other served as the traffic control coordinator, monitoring other vehicles in the vicinity of the test course. This person also had the responsibility of assuring that the water system was functioning properly. An additional person was needed to drive the water truck for the 55 and 70 mph braking tests. The water truck made two passes on the course after every two braking maneuvers, but it was the responsibility of the test administrator in the vehicle to determine if additional water was needed. Two-way radio communication was maintained between all personnel.

Test Procedure. Each of the test subjects were instructed to drive the vehicle through the test course at the required test speed. The first series of tests for each test subject were the *anticipated* maneuvers. The test administrator counted down by saying "Ready, set..." and then illuminated the windshield-mounted signal. When the test subject saw the signal, they were instructed to bring the vehicle to a stop as quickly as possible, without deviating from the 12-foot wide lane. Three trial runs were performed for each test condition (speed, pavement, geometry, and ABS). The second series of tests for each test subject were the *surprise* maneuvers. Each test subject was instructed that somewhere along the test course the signal would illuminate. At the onset of this *surprise* signal, they were to bring the vehicle to given, thereby deliberately minimizing their expectancy. For eac test condition, five trials were performed.

Subject I.D. OF1 Run No. 6 Date 01-20-1994

TIME	DIST	V	A	M	T	B	Gx	Gy	
0.010	0	0	0	0	1	0	0.03	0.06	
0.020	0	0	0	0	1	0	0.04	0.08	
0.030	1	0	0	0	1	0	0.03	0.09	LED ON
0.040	2	0	0	0	1	0	0.03	0.07	
0.050	3	0	0	0	1	0	0.03	0.06	
0.060	4	0	0	0	1	0	0.01	0.07	Throttle OFF
break									
0.640	52	0	0	0	1	0	0.01	0.06	
0.650	52	0	0	0	1	0	0.02	0.05	
0.660	53	1	0	0	1	0	0.04	0.04	
0.670	54	1	0	0	1	0	0.05	0.03	
0.680	55	1	0	0	1	0	0.04	0.05	
break									Brake ON
1.050	86	1	0	0	1	0	0.03	0.02	
1.060	86	1	0	0	1	0	0.03	0.02	
1.070	87	1	0	0	1	0	0.04	0.04	
1.080	88	1	0	0	0	0	0.04	0.05	
1.090	89	1	0	0	0	0	0.04	0.06	
break									
1.180	96	1	0	0	0	0	0.03	0.04	
1.190	97	1	0	0	0	0	0.02	0.03	
1.200	98	1	0	0	0	1	0.02	0.04	
1.210	99	1	0	0	0	1	-0.00	0.05	
1.220	100	1	0	0	0	1	0.00	0.05	
1.230	101	1	0	0	0	1	0.02	0.04	GxMax
break									
1.570	128	1	0	0	0	1	-0.63	0.11	
1.580	129	1	0	0	0	1	-0.64	0.09	
1.590	130	1	0	0	0	1	-0.66	0.07	
1.600	131	1	0	0	0	1	-0.67	0.09	
break									
5.800	275	1	0	0	0	1	-0.20	0.03	
5.810	276	1	0	0	0	1	-0.20	0.03	
5.820	276	1	0	0	0	1	-0.20	0.03	
5.830	276	1	0	0	0	1	-0.20	0.03	
5.840	276	1	0	0	0	1	-0.19	0.05	Stop Distance
5.850	276	1	0	0	0	1	-0.19	0.05	
break									
6.970	276	1	0	0	0	1	0.01	0.03	
6.980	276	1	0	0	0	1	0.01	0.02	
6.990	276	1	0	0	0	1	0.00	0.03	
7.000	276	1	0	0	0	1	0.02	0.02	
7.010	276	1	0	0	0	1	0.00	0.00	
7.020	276	1	0	0	0	1	0.00	0.00	
7.030	276	1	0	0	0	1	0.00	0.00	

FIGURE C-2. Typical Data File for One Braking Maneuver.

Study 2 – Closed-Course Braking Study: Volunteer Drivers, TTI Vehicles

Background. After Study 1, several variables, or test conditions, were eliminated because of non-significant performance results. The research could then focus on variables that did produce significant differences and the behaviors of the drivers that produced these results. Because similar deceleration characteristics were found between *anticipated* versus the *surprise* maneuvers in Study 1, the *anticipated* variable was eliminated from the remaining studies. The *surprise* condition is more closely associated with real-world stopping conditions and was used for all but a few practice runs in subsequent studies. There were few differences in braking distances at 40 mph for ABS versus no ABS, but significant differences resulted at 55 mph. Also, no differences existed between braking on a left or right horizontal curve.

Test Subjects. Study 2 required a group of volunteer test subjects more representative of the driving population than the group of subjects in Study 1. Therefore, persons in the local area, including those from the active TTI participant list, students from Texas A&M University, and senior citizens from a local church group, were contacted by telephone to determine their interests in this study. A total of 26 subjects were scheduled for participation. A breakdown of this group included 12 in the younger group and 14 in the older group. Of the 12 in the younger group, six were males and six were females. In the older group, seven were males and seven were females.

Test Conditions. All 26 test subjects performed braking maneuvers at an initial speed of 55 mph. They were tested with the following test conditions: with enabled or disabled antilock brakes, wet or dry pavement conditions, and on two geometric conditions, including a tangent section and a horizontal curve section. The antilock brake variable (either on or off) remained constant throughout the testing for one particular test subject, as opposed to the test conditions in Study 1 where each test subject performed with and without ABS. The test subjects in Study 2 were not informed whether or not ABS was enabled. If the test subject inquired, the test administrator simply stated "the vehicle is equipped with ABS but it may or may not be working".

Study 2 included three different parts: an unexpected object segment, an expected object segment, and a simple reaction time test. The unexpected object segment, Part A, was different from the *surprise* condition in Study 1. The initial portion of this study required test subjects to perform a maneuver to a truly unexpected object encountered in the roadway, similar to AASHTO's stopping sight distance assumptions. Since an unsuspecting driver can only be truly surprised once in a testing environment, each test subject provided a single data point. Part B of this study continued after this initial maneuver, and test subjects performed a series of *anticipated* braking maneuvers to an expected object condition, similar to the *surprise* condition in Study 1. They stopped their vehicle at the onset of the windshield mounted LED. Part C required each test subject to perform a simple reaction time test while sitting in the driver's seat of the test vehicle. Furthermore, a separate group, or a control group, of test subjects that did not participate in Parts A and B also performed the simple reaction time test. All of the above conditions for Study 2 are summarized in Table C-14.

Course Layout. Study 2 was also conducted at the Texas A&M University Riverside Campus on a closed-course test track. Based on findings from Study 1, this study was conducted at an initial speed of 55 mph. Therefore, to utilize the existing watering system on the 40 mph course and because the Apron 1 area was an ideal location to design the unexpected object scenario, a new course was designed. This course was surveyed using the 55 mph design side friction value previously presented in Table C-12. A detailed view of this course is shown in Figure C-3.

The new 55 mph test course was also an "S" curve bounded by two tangents at each end. The course simulated a two-lane roadway, with 12-foot wide lanes, delineated by a yellow centerline stripe and orange traffic cones representing the pavement edge. The solid white pavement edge stripes had to be removed because of the new design. Similar to the 40 mph course in Study 1, test subjects entered on a tangent from the south, made a curve to the left (Curve 1) and then transitioned to a second curve to the right (Curve 2) before exiting to another tangent section at the north end of the course.

The southern most tangent (Tangent 1) was part of Taxiway 7 and provided adequate distance for braking maneuvers. At the point-of-curvature of Curve 1 (PC_1), the yellow centerline continued on Curve 1 and a white centerline continued on the tangent (Tangent 2). Basically, the course diverged at PC_1, marked by two different-colored centerlines.

The watering system was positioned to distribute water on the first 300 feet of Curve 1 and on Tangent 2. Curve 1 was the 55 mph design curve. It was 740 feet in length, and the point-of-tangency (PT_1) was the point-of-curvature of Curve 2 (PC_2). This second curve was 390 feet in length, and at PT_2, a 200 foot tangent (Tangent 3) led up to a point where the unexpected object was placed. Curve 2 was not a design curve. Although its degree of curvature was slightly higher, it still traversable at 55 mph. It was retrofitted between Curve 1 and Tangent 3 because of fixed objects located on the Apron 1 area, particularly a large friction pad used for other vehicle performance studies. Curve 2 was not used for braking maneuvers.

Similar to Study 1, a small holding pond in Area 10 was used to obtain water for wet pavement braking maneuvers. The water system layout was slightly modified from the previous study in order to distribute water to a portion of Curve 1 and enough distance on Tangent 2 to perform wet pavement braking maneuvers. Because of limited power from the water pump, the system could not be positioned to distribute water to a portion of Tangent 1 and Curve 1 simultaneously; thus the need for the Tangent 2 layout.

TABLE C-14. Summary of Test Conditions for Study 2.

	Number of Conditions Per Test Subject					
Segment - Condition	Speed *(mph)*	ABS	Pavement *(Wet/Dry)*	Geometry *(Tangent/Curve)*	# Trials	Total
Part A - Unexpected	55	Off	Dry	T	1	1
Part B - Expected	55	Off	Wet/Dry	T/C	5	20
Part C - RT Test					10	10
Part D - RT Test (control)					10	10
Total						41
Total Maneuvers = 26 Test Subjects x 31 Trials						986

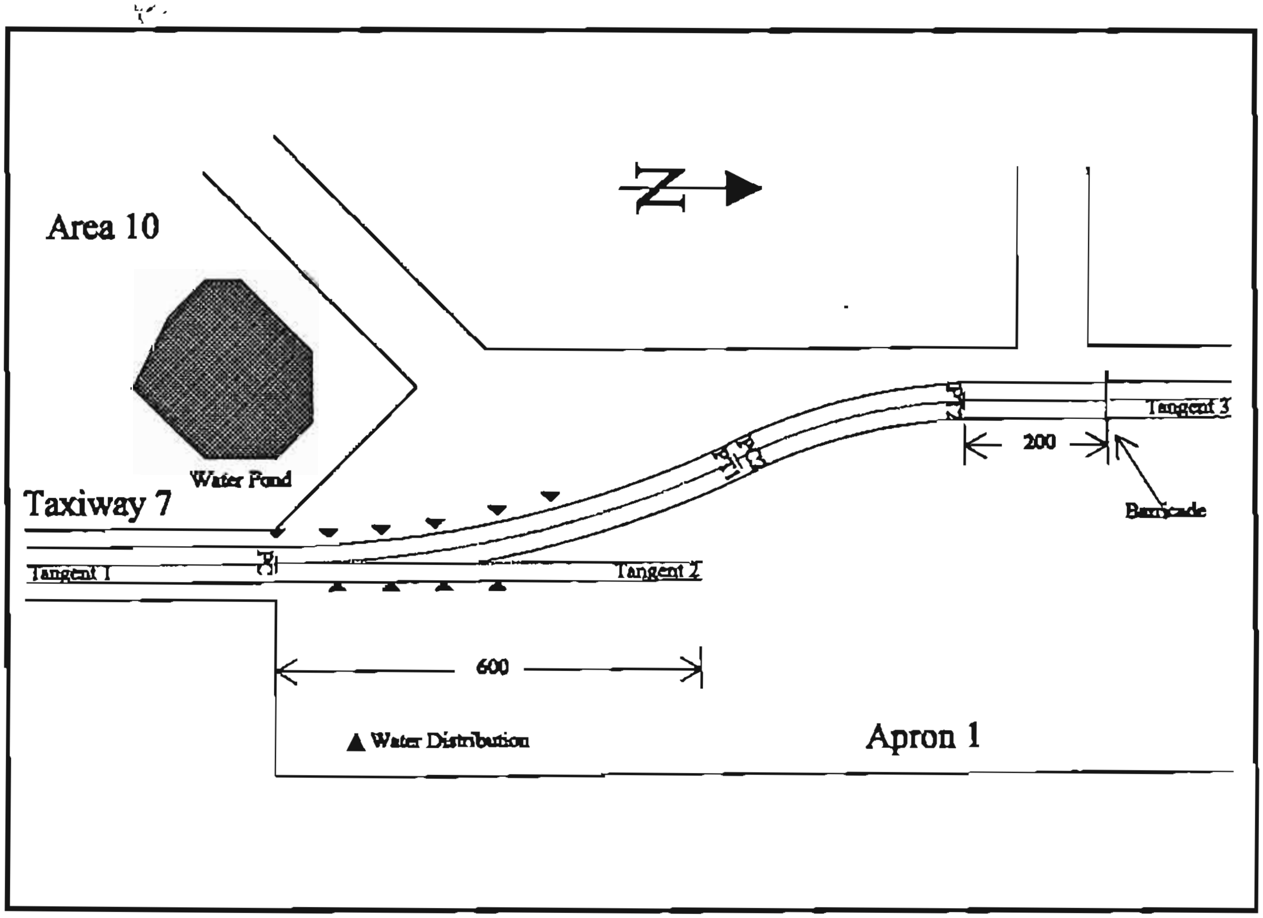

FIGURE C-3. Test Track Curve Layout for Study 2.

was located on Tangent 3, at the north end of the course, 200 feet north of PT_2. The unexpected object was a 3-foot high fabric barricade, spanning both lanes of the roadway, that suddenly appeared in the path of the driver. It was initially stored in a small, 2-inch wide trench cut into the concrete pavement and covered by black rubber strips to prevent it from being seen by the test subject. The fabric was a lightweight, black landscaping material, and attached to the material were four 36 by 36-inch octagon-shaped Stop signs (white on red), made of a heavier polyester material. This barricade was designed to represent an object that would suddenly appear over the crest of a vertical curve and compel the driver to stop the vehicle before hitting it. The barricade was, of course, designed to break-away if struck by the vehicle. The test subject's performance was recorded as they responded to this scenario.

The unexpected object appeared 210 feet in front of the test subject. This value was based on the stopping sight distance equation, using a 1.0 second perception-brake reaction time time and a dry pavement friction value of 0.80. The research team decided that if the subject was provided more than 1.0 second of PBRT time, he or she would have time to think of an appropriate evasive maneuver. It was the intent of this study was not to allow time for an evasive maneuver, but to have the test subject *stomp* the brakes to stop the vehicle before striking the barrier. Stopping before striking the barrier was secondary to having the subjects react to this unexpected object. A photograph of this barrier is shown in Figure C-4.

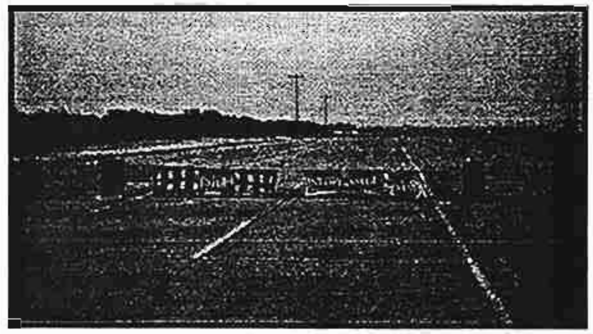

FIGURE C-4. Unexpected Object Used in Study 2.

Test Vehicle and Instrumentation. The same vehicle used for Study 1, a 1991 four-door Chevrolet Caprice sedan from the Texas A&M University motor pool, was again used for all subjects in Study 2. The same maintenance and inspection routine was applied to this study to assure reliable performance. The antilock braking system was either enabled or disabled for each test subject (but not both), and they were not informed whether or not the vehicle had or did not have ABS.

The same instrumentation also remained in the vehicle for Study 2. The only modification was the addition of a radio frequency transmitter in the vehicle to send a signal to initiate the unexpected barrier. A receiver hidden from the test subject's view was located near the barrier to receive the radio signal. A pressurized tank mechanism provided the force to *spring* the barrier. In the vehicle, the *mark* button on the hand-held control pendant was used by the test administrator to deploy the barrier. There was a 0.27 second delay from the moment the *mark* button on the pendant was pushed until barrier movement could be detected. The total delay to full deployment was 0.5 seconds.

Test Personnel. Two test personnel were required for this study. One person served as the test administrator in the vehicle and the other served as the traffic control coordinator, monitoring other vehicles in the vicinity of the test course. This person also had the responsibility of assuring that the water system was functioning properly. Communication between both personnel was maintained via two-way radios.

Test Procedure. The test procedure consisted of four parts. Each part is discussed in the following sections:

Part A — Unexpected Object Segment. Each test subject initially signed a consent form and was then instructed to drive out to the test site. Once at the site, the test subject was instructed to drive in a normal manner on the closed course at 55 mph, accompanied by a test administrator in the vehicle. Each subject initially drove through the course approximately three to four times to get accustomed to the pavement markings and cones that mark the course. No specific instructions were provided other than assurance that nothing could harm them in the closed course, provided they stayed in their lane. After feeling comfortable driving the course at 55 mph, the test subject was instructed to stop at the south end of the course while the test administrator radioed the traffic controller to request that the vehicle instrumentation be *"calibrated."* The *calibration* command provided a cue to the traffic controller to remove the rubber strips from the trench that concealed the barrier.

The subject was then instructed to drive through the course one more time, at 55 mph, to *"calibrate"* the instrumentation. As the subject approached the north end of the course and Tangent 3, the barricade was deployed. The subject's performance was recorded as they performed the surprise stopping maneuver. This one-time-per-driver maneuver served as an introduction to the next part of the study.

Part B — Expected Object Segment. After the initial maneuver, the test subjects were then provided additional information about the study and instructed to drive along the course again at 55 mph. They were then instructed to bring their vehicle to a stop at the onset of the windshield-mounted LED. The only requirement stated to the test subject was that they were to bring the vehicle to a stop as quickly as possible, but to also stay within the 12-foot lane. The anticipated surprise performance can be contrasted with the performance in the unexpected-object portion of the study. Similar to what Johansson and Rumar did in their research, an adjustment factor could be determined for *surprise* and *anticipated* perception-brake reaction times (*12*). Furthermore, *surprise* and *anticipated* deceleration profiles provide insight into how drivers perform during a braking maneuver and what type of profile they exhibit, especially when confronted with an unexpected object versus *anticipatory* stops.

The braking maneuvers for each test subject were performed on two different geometric conditions, a tangent and a curve; and two different pavement conditions, wet and dry. The dry braking maneuvers were conducted first, and maneuvers were conducted in both directions on the course. Only one braking maneuver was performed each time through the course. After a braking maneuver, the test subject proceeded to the other end, turned the vehicle around, and proceeded in the opposite lane for the next maneuver. After 10 repetitions, the watering system was turned on and the wet pavement braking maneuvers were conducted. The wet curve braking maneuvers could be performed in either direction, but the wet tangent braking maneuvers could only be performed while the vehicle was in the north-bound lane and on Tangent 2 (see Figure C-3). Five braking maneuvers were performed for each condition, for a total of 20 test runs. Crash helmets were worn by the subject and the test administrator for all braking maneuvers.

Part C – Simple Reaction Time Test, Volunteer Drivers. This part of the study was conducted to determine a driver's reaction time and foot-movement time when presented with an anticipated stimulus in a closed environment. The reaction and foot-movement testing conducted in this study was considered a *baseline* test, representing the fastest response performance a driver can physically achieve. Each test subject performed ten simple reaction time tests while sitting in the driver's seat of the test vehicle. While the vehicle's transmission was in park, the test subject was asked to place his foot on the throttle and then depress the brake whenever the windshield mounted signal was illuminated. This simple reaction time test was conducted immediately after the dry pavement test runs, while the course was being wet down, and a total of ten repetitions were conducted.

Part D – Simple Reaction Time Test, Church Group. This study was conducted to demonstrate the differences between the group of test subjects that participated in Studies 2, 3, and 4 a separate control group, if any. The pool of older test subjects used in the driving portions of Studies 2, 3, and 4 may have demonstrated biased reaction performance characteristics because of the anticipation of being in an experimental situation and/or because of their active participation in other empirical studies. Therefore, a simple reaction time test was conducted with a control group of test subjects, a group of older drivers from a local church, that have never before participated in driving performance tests or other related empirical research. The control group presumably did not enter any type of biases that the test subjects in Studies 2, 3, and 4.

This study required a group of older volunteer subjects representative of the driving population. The only exception to the previous studies was to find a group of drivers who had not participated in research-related studies, especially studies requiring a response or driving performance. A total of 18 persons participated in this study, all from a local senior citizen church group. Of the 18 persons, ten were females and eight were males ranging from 60 to 83 years old.

For each test subject, ten reaction time tests were conducted in a parked TTI vehicle (a 1991 Ford Crown Victoria), requiring their response to the windshield-mounted LED signal. For each subject, seat adjustments were made for their comfort and, if necessary, the LED was adjusted to fall in their horizontal field of view. Each test subject participating in this study was required to sit in the driver's seat of the vehicle. The vehicle remained stationary throughout the test while the subject's reactions were measured. Similar instrumentation was required, as before, to measure foot placement on the throttle and brake. The QuickBASIC program on the Compaq computer managed the other hardware components that were necessary, which included the windshield-mounted LED, the control pendant, and the foot-sensitive hardware.

Two members of the research team traveled in the instrumented vehicle to the church to meet with this group of subjects. A brief explanation was provided to the group concerning the purpose of the study, and simple instructions were provided once each subject was in the vehicle. Each subject was instructed to sit in the driver's seat, place their right foot on the throttle, and then depress the brake as quickly as they could, with the same foot, whenever the windshield mounted signal was illuminated. For each subject, ten repetitions were conducted, which took approximately 10 minutes.

Study 3—Closed-Course Braking Study: Volunteer Drivers, Personal Vehicles

Background. Study 3 was conducted to determine driver/vehicle braking performance characteristics of drivers operating in their own vehicle, rather than a vehicle owned and maintained by someone else. This study replicated Study 2 with the only difference being the type of vehicle being tested.

Test Subjects. Study 3 required a group of volunteer subjects representative of the driving population. Therefore, persons in the local area, including those from the active TTI participant list and students from Texas A&M University, were contacted by phone to determine their interests in this study. A total of 12 subjects participated. A breakdown of this group included five in the younger group and seven in the older group. Of the five in the younger group, three were females and two were males. Of the seven in the older group, four were females and three were males.

Test Conditions and Course Layout. Similar to the conditions of Study 2, all braking maneuvers were conducted at a speed of 55 mph, including the unexpected object segment. The only major difference in this study was the fact that none of the personal vehicles driven had antilock braking systems, which will be noted in the data analysis. The study again included three different parts: an unexpected object (surprise) segment, an expected object (anticipated) segment, and a simple reaction time test. All of the test conditions for Study 3 are summarized in Table C-15. Study 3 was also conducted at the Texas A&M University Riverside Campus. The same closed-course track used for Study 2 is again used for Study 3 (see Figure C-3).

Test Vehicle and Instrumentation. The subjects participating in this study each drove their own vehicle during the testing. They were notified prior to scheduling that this study would require them to drive their own vehicle. The same instrumentation used previously was modified to be installed in each of these vehicles. The only major change was the substitution of a Datron, non-contact fifth-wheel for the standard fifth-wheel device which was attached to the bumper of the TTI vehicle in Studies 1 and 2. The Datron provided the same information as the bumper-attached device, but could be installed on any vehicle. A suction-cup mounting bracket held the small Datron device, which attached to the passenger side door. All other power and serial port connections remained the same, only more compact for mobility. A listing of the personal vehicles driven in this study is presented in Table C-16. Note that two married couples participated in this study and drove the same car. Thus, only 10 different cars were used in Study 3.

Test Personnel. Two test personnel were required for this study. One person served as the test administrator in the vehicle and the other served as the traffic control coordinator, monitoring other vehicles in the vicinity of the test course. This person also had the responsibility of assuring that the water system was functioning properly. Two-way radio communication was maintained between both personnel.

Test Procedure. Study 3 test procedure consisted of three parts. Each part is discussed in the following sections:

Part A — Unexpected Object Segment. The administrator in the vehicle instructed the test subject to sign a consent form and to drive out to the test site. The test subject was then instructed to drive in a normal manner on the closed course at 55 mph. Similar to the previous study each test subject initially drove through the course approximately three to four times to get accustomed to the pavement markings and cones that marked the course. After maintaining the 55 mph test speed, the test subject was then instructed to drive through the course one more time to *"calibrate"* the instrumentation. As the test subject approached the north end of the course and Tangent 3, the barricade was deployed. The subject's performance was recorded for this *surprise* braking maneuver.

Part B — Expected Object Segment. After the initial maneuver, the test subject was then provided additional information about the study and was instructed to drive along the course again at 55 mph. They were then instructed to bring their vehicle to a stop at the onset of the windshield-mounted LED. The only requirement stated to the driver was to bring the vehicle to a stop as quickly as possible, but to also stay within the 12-foot lane. The same test conditions applied as in Part B of Study 2.

Part C — Simple Reaction Time Test. Each driver performed simple reaction time tests while sitting in the driver's seat of their vehicle. While the vehicle's transmission was in park, the driver was asked to place his right foot on the throttle and then depress the brake, with the same foot, whenever the windshield mounted signal was illuminated. This simple reaction time test was conducted immediately after the dry pavement test runs, while the course was being wet down, and a total of ten repetitions were conducted.

TABLE C-15. Summary of Test Conditions for Study 3.

	Number of Conditions Per Test Subject					
Segment - Condition	Speed *(mph)*	ABS	Pavement	Geometry *(Tangent/Curve)*	# Trials	Total
Part A - Unexpected	55	Off	Dry	T	1	1
Part B - Expected	55	Off	Wet/Dry	T/C	5	20
Part C - RT Test					10	10
Total						31
Total Maneuvers = 12 Test Subjects x 31 Trials						372

TABLE C-16. Personal Vehicles Used in Study 3.

Year	Make	Model	# Doors	Vehicle Type	ABS
1982	Mazda	B-2000	2 Door	Light Truck	No
1983	Toyota	Corolla	2 Door	Passenger Car	No
1985	Cadillac	El Dorado	2 Door	Passenger Car	No
1985	Honda	Prelude	2 Door	Passenger Car	No
1987	Oldsmobile	Cutlass Supreme	2 Door	Passenger Car	No
1987	Pontiac	Firebird	2 Door	Passenger Car	No
1988	Oldsmobile	88 Royale	4 Door	Passenger Car	No
1989	Oldsmobile	88 Royale	4 Door	Passenger Car	No
1991	Chevrolet	Blazer	2 Door	Sport Utility	No
1992	Ford	Taurus	4 Door	Passenger Car	No

Study 4--Open-Road Braking Study: Volunteer Drivers, Personal Vehicles

Background. Study 4 was conducted to determine a driver's perception-brake reaction times when presented with an unexpected object on the open road. Unlike the methods used in Studies 2 and 3 when drivers were presented with an unexpected object in a closed-course environment, this study's experimental approach required participants to drive to a rural, low volume, two-lane roadway, in their own vehicle, accompanied by a test administrator in the vehicle. Each test subject was led to believe that they were not involved in a driver performance test, but rather, a roadway evaluation test. Once at a particular site on this roadway, an unexpected object suddenly appeared in their field of view and moved toward them from the right side of the roadway. The intent of the study was not to have the subject brake to a complete stop to this object, but rather, react to the unexpected object by depressing the brake, making a steering maneuver, or both, and then drive on after realizing that the object could be avoided.

Test Subjects. Study 4, similar to Studies 2 and 3, required a group of volunteer subjects representative of the driving population. Therefore, persons in the local area, including those from the active TTI participant list and students from Texas A&M University, were contacted by phone to determine their interests in this study. A total of 12 subjects participated in Study 4. A breakdown of this group included six in the younger group and six in the older group. In each group of six, three were males and three were females.

Test Conditions. No preset test conditions, such as a particular speed or the required use of antilock brakes, applied to this study. For each test subject only one test run was conducted, and the primary interest was the test subject's reaction to the unexpected object. No other test conditions applied for the study other than the requirement of dry pavement conditions. All test runs were conducted at a speed of approximately 45 mph, but varied slightly according to the test subject's preference.

Course Layout. Study 4 was conducted along a portion of Old San Antonio Road (OSR) near Bryan, Texas, approximately three miles from the Texas A&M Riverside Campus. OSR is a state-maintained, rural, two-lane roadway with an ADT of approximately 300 vehicles per day and a posted speed limit of 55 mph. The site along this roadway where the unexpected object appeared was in the middle of an approximately one-mile, flat tangent section. A layout of the test section is shown in Figure C-5.

Test Vehicle and Instrumentation. The test subjects participating in this study each drove their own vehicle during the testing. Subjects were notified prior to scheduling that this study would require them to drive their own vehicle. The same instrumentation used previously was installed in each person's vehicle for this study. Table C-17 lists the vehicles that were driven for this study. Note that four married couples participated in Study 4. Thus, only eight different vehicles were used.

The unexpected object used in this study was a 100-gallon, 30-inch diameter, empty cardboard barrel. The barrel rolled down a ramp attached to the back of a pick-up truck. This truck was parked perpendicular to, and on the right side of the roadway, with the rear of the truck about five feet from the edge of the pavement. A mechanism in the back of the truck released the barrel when a radio frequency signal was activated by the test administrator in the vehicle. A photograph of this scenario is shown in Figure C-6.

TABLE C-17. Personal Vehicles Used in Study 4.

Year	Make	Model	# Doors	Vehicle Type	ABS
1978	Toyota	Truck	2 Door	Light Truck	No
1983	Volvo	240	4 Door	Passenger Car	No
1986	Mazda	300-ZX	2 Door	Passenger Car	No
1991	Ford	Mustang	2 Door	Passenger Car	No
1992	Cutlass	Sierra	4 Door	Passenger Car	No
1992	Toyota	Camry	2 Door	Passenger Car	No
1994	Lincoln	Continental	2 Door	Passenger Car	Yes
1994	Mercury	Sable	4 Door	Passenger Car	Yes

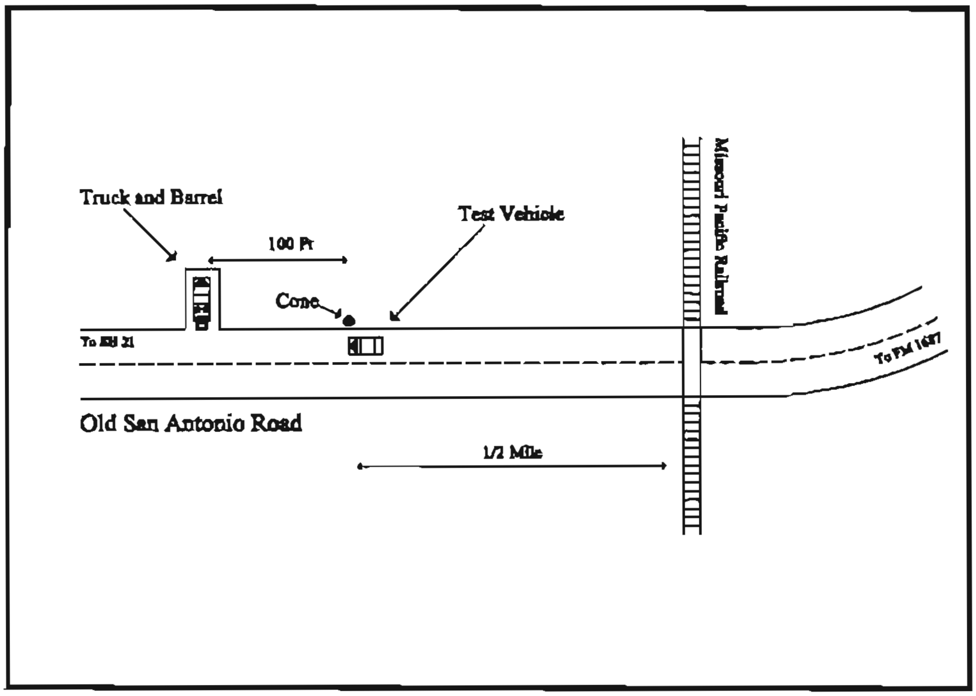

FIGURE C-5. Test Section Layout for Study 4.

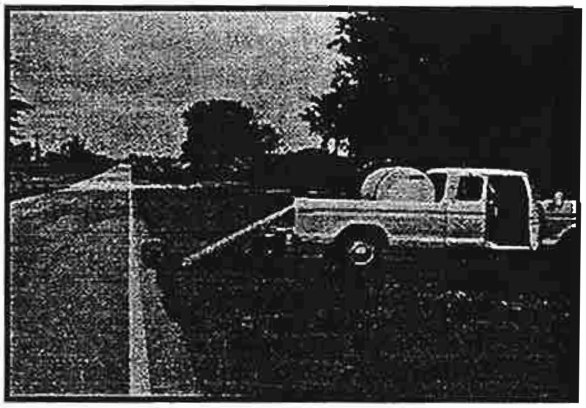

FIGURE C-6. Unexpected Object Scenario Used In Study 4.

Similar to the experimental approach to the unexpected object in Studies 2 and 3, it was decided that if the subject was provided with more than 1.0 second of perception-brake reaction time time, he or she would have time to think of an appropriate evasive maneuver. It was the intent of this study to not allow time for an evasive maneuver, but rather have the test subject take whatever action they might take after perhaps realizing that they could not stop before striking the object. Approaching at approximately 45 mph, the barrel appeared 75 feet in front of the test subject. At this point, it was at the top of the ramp and was rolling towards the travel lane. The situation was designed to appear as if the barrel would strike the front of the vehicle if the vehicle was approaching at 45 mph. Once the barrel reached the bottom of the ramp and approached the roadway, it was snagged by small cords attached to the barrel. The cords prevented it from rolling any further than the edge of the pavement.

Test Personnel. Two test personnel were required for this study. One person served as the test administrator in the vehicle and the other person stayed with the truck from which the barrel rolled. This person impersonated a worker or farmer on the side of the road, rolling other barrels around the vicinity of the truck as the test vehicle approached. Because of this site's location, this truck, the *worker*, and the barrels blended in with the rural setting. Other vehicles in the vicinity were monitored by the test administrator in the vehicle.

Test Procedure. Each test subject first signed a consent form at the TAMU Riverside facilities. Accompanied by the test administrator, the test subject was then instructed to proceed out to OSR. In order to make the subject believe that this study was something other than a driver performance study, the subject was instructed to pay attention to the roadway conditions, such as the pavement condition, the signs, and the roadside features. They were told that this roadway evaluation was necessary because the Texas Department of Transportation (TxDOT) would eventually be upgrading the road based upon the recommendations of this study. Because of a major construction project near the TAMU Riverside Campus and the relatively poor roadway conditions of OSR, each of the subjects found this to be a credible study.

Once the subject reached OSR, they were instructed to drive at a speed that they felt comfortable driving. The subject was also instructed to observe the conditions of the road, and to be prepared to answer subjective questions about the roadway conditions on the return trip to the TAMU Riverside Campus. After driving approximately three miles, the subject passed the truck, the *worker*, and the barrels on the left side of the roadway. The test subject proceeded one mile further, and was then instructed to turn the vehicle around and to proceed back to the TAMU Riverside Campus. The subject was also instructed to drive at a more *moderate* speed on the return trip. No specific vehicle speed was stated to avoid the possibility of the test subject concentrating on traveling at a particular speed. Furthermore, if the test subject was required to travel at a certain speed, he or she would have possibly overcompensated because the approach was on a downgrade. The downgrade slightly increased the speed, and if a specific speed would have been required, the test subject would have removed his or her foot off of the throttle before the unexpected object actually appeared, rendering the data useless.

As the test subject approached the truck on the right side of the road, the barrel suddenly appeared before them and rolled towards the travel lane. The test subject's performance (throttle release, brake depression, and lateral acceleration) was measured to this unexpected object. Once beyond this point, the test administrator instructed the test subject to stop the vehicle and told the subject the actual purpose of the test. A few subjective questions concerning unexpected object conditions were asked and then the test subject returned to the TAMU Riverside Campus.

RESULTS - STUDY 1

Study 1 was conducted at the Texas A&M University Riverside Campus on closed-course conditions, and was a preliminary study that measured braking performance characteristics of a group of *average* and *expert* drivers for a variety of test conditions. This section is separated into four major sections. The first section presents the findings of the driver's perception-brake reaction times to anticipated and surprise conditions. The second section presents a discussion on reaction and foot movement times with respect to to the different test conditions. The third section presents the results of the driver's braking distance performance. The fourth section presents the deceleration characteristics.

Perception-Brake Reaction Times

This analysis focused on the reaction time and foot-movement time (from throttle to brake) of the test subjects in the vehicle. Reaction time was obtained by subtracting the time of LED onset from the time of release of the throttle pedal as the driver begins his or her braking maneuver. Foot-movement time was obtained by similarly subtracting time of throttle pedal release from time of brake pedal onset.

Since there exist a number of runs in which brake pedal and throttle pedal events were not adequately captured on the file record obtained on each run, it was decided to exclude any reaction time values in excess of 3.0 seconds and any foot-movement values greater than 1.0 second as outliers. This scheme resulted in 1,324 observations for reaction time and 1,212 observations for foot-movement time from a possible 2,088 observations.

Eight test subjects were used in this analysis, under conditions of both unexpected and expected object conditions to the onset of the LED signal. Data for nominal speeds of 40 and 55 mph were used. Only three test subjects, the *expert* drivers, were tested at the 70 mph test speed, and that data was not included in this analysis. Since this analysis was concerned solely with driver responses prior to the braking event, data was collapsed over pavement, geometry, and ABS conditions.

The reaction and foot-movement data for one of the TTI subjects were excluded from this analysis because the subject was a *left-footed* braker. Foot-movement time for a test and subject performing the braking maneuver with the left foot could not be calculated because of the arrangement of the instrumentation in the test vehicle. The reaction time was obtained, but was considerably less than the remaining eight test subjects.

Reaction And Movement Times

Overall reaction time averaged across all conditions was 0.34 seconds, with a standard deviation of 0.17 seconds. Overall average movement time was 0.18 seconds, with a standard deviation of 0.094 seconds. Neither the main effects of *Stopping Condition, Speed,* or the interaction of *Condition* was statistically significant for reaction time. Foot-movement time was significantly slower (0.208 seconds) for maneuvers under the *anticipated* conditions, as compared to 0.168 seconds under the *surprise* conditions. When drivers did not know when to expect the onset of the LED, their leg movement was slightly faster. Neither *Speed* nor the interaction of *Stopping Condition and Speed* were significant. A 95 percent confidence interval was established for this data to estimate the probability that a similar study run with the same number of test subjects and trials would yield mean values for reaction time and movement time within these confidence limits. The means and confidence intervals are presented in Table C-18.

Because the distribution of these data is not strictly normal, but rather is positively skewed with a cutoff of zero, a log transformation was used on the data and the analysis of variance models (ANOVA) were again tested. Log transformations tend to *normalize* the data and better satisfy the assumptions underlying the use of ANOVA. Results were similar to untransformed data, although differences between *anticipated* and *surprise* reaction times also were statistically significant. There is a weak tendency for *anticipated* responses to take longer than *surprise* responses.

Using the Taoka (*33*) conversion of data to a lognormal model to estimate percentiles of reaction and movement time, the estimates of reaction and movement time in Table C-19 can be made. A *worst case* driver at a 99 percentile would take 0.93 seconds to react to an obstacle or other emergency in the roadway, and an additional 0.50 seconds to move his foot from the throttle pedal to the brake pedal. This sum would add to a total perception-brake reaction time of 1.43 seconds, as compared to the current AASHTO value of 2.5 seconds.

TABLE C-18. Confidence Intervals for Reaction and Movement Time.

	Condition	95% Confidence Interval (seconds)
Reaction Time	All Conditions	0.34 ± 0.01
Movement Time	Anticipated	0.21 ± 0.01
	Surprise	0.17 ± 0.01

TABLE C-19. Percentile Estimates of Reaction and Movement Time.

	Reaction Time (seconds)	Movement Time (seconds)
Mean	0.30	0.16
85th	0.50	0.27
95th	0.67	0.36
99th	0.93	0.50

Braking Distances

It should be emphasized that the reported braking distance data are based upon the *nominal* rather than the actual velocity of the vehicle at the onset of the braking maneuver. Researchers performing this analysis decided to accept variation in initial velocity as error variance. Any files that showed more than a 3 mph difference between velocity and nominal were eliminated from this analysis.

Findings for 40 and 55 Miles Per Hour. An ANOVA test procedure for the dependent variable *braking distance* was conducted for the 40 and 55 mph test data. Five independent variables, including *Stopping Condition, Geometry, ABS, Pavement*, and *Speed* (plus *Subjects*, since the model is a fixed-effects within-subjects or repeated-subjects design) were considered. The variable *Speed* had two levels for this analysis: 40 and 55 mph. Only the main effects were considered in this model. Interaction of *Subject* with each of the four variables of interest form the error terms for the F test ratios.

Neither the *Stopping Condition* nor the *Geometry* levels of those variables were significantly different from one another; test subjects achieved about the same braking distance under a *surprise* condition as they did when they were counted down to a braking response. Test subjects performed about as well on tangents as on horizontal curves, either to the right or left. As expected, both pavement conditions and the nominal speed prior to the maneuver showed a high level of significant difference in braking distance. The ABS condition also resulted in a significant differences in braking distance. Figure C-7 shows the braking distance comparisons between wet and dry conditions, and with and without ABS at the three nominal test speeds. Note the differences between wet and dry pavement and the relatively small benefits of ABS at 40 mph. The figure shows identical results with much larger differences between ABS versus no ABS (on the order of 50 feet) at 55 mph.

The subject groupings under the Duncan multiple-range test did not follow the expectation that the *expert* TTI drivers (three men with racing and driver training backgrounds) would form a group different than the six other TTI drivers. The six TTI drivers were comprised of three young male students and three males over the age of 55. The results of the Duncan Groupings are listed in Table C-20.

Findings for 70 Miles Per Hour. The 70 mph nominal speed data were analyzed separately from the 40 and 55 mph data because only three of the nine test subjects participated in these high speed runs, whereas all nine were tested at 40 and 55 mph. The ANOVA test for these data was identical in format to the 40 and 55 mph test except that it dropped the variable of *Speed*.

From the analysis, very weak significance levels were observed for the main effects, due to the small number of test subjects. Only the variable *ABS* reached any kind of respectable level. High variance in these data were observed due to the braking distance disparity of the three expert drivers. Test Subjects E1 and E3 had mean braking distances 324 and 343 feet, respectively. Test Subject E2, however, had a mean braking distance of only 240 feet. Test Subject E2 was not tested for any 70 mph maneuvers under wet pavement conditions due to scheduling conflicts, which accounts for this disparity. Figure C-7 illustrates the findings, with the same trends evident as for the 40 and 55 mph data. At 70 mph on wet pavements, about 90 feet of distance is saved if ABS is enabled.

TABLE C-20. Duncan Groupings for Study 1 Drivers.

Duncan Grouping[1]	Mean Braking Distance (feet)	Sample Size N	Subject I.D.[2]
A	182.2	150	T1
B	156.2	202	E1
B	151.1	174	T5
C	138.4	189	E3
C	137.3	160	T7
C	135.5	142	T8
C	135.5	141	T9
C	131.8	191	T6
D	109.2	150	E2

[1] *A,B,C,D are Duncan Groupings. Items with same letter are not significantly different within the group*
[2] *T = Ordinary (TTI) Driver, E = Expert Driver*

Decelerations

Maximum Deceleration. An identical analysis as those reported previously was performed for the variable *Max* G_x, the maximum deceleration achieved during a particular braking maneuver. The reader should note that *Max* G_x is not equivalent to sustained deceleration for the entire maneuver. The deceleration profile (Figure C-8) typically yields a maximum value at some time during the maneuver (in this case, at 4.2 seconds) and then falls off or fluctuates at a lesser negative value until the vehicle is at a standstill. Note that this curve is anything but linear.

From the summary for the ANOVA tests of the 40 and 55 mph data, the independent variables *ABS* and *Pavement* levels were significant; note that maximum deceleration is not sensitive to nominal velocity. Drivers at 40 and 55 mph exhibited the same maximum deceleration regardless of the speed at the initiation of the braking maneuver. Drivers were, however, very divergent in what their maximum deceleration performance was. The *Condition* and *Geometry* variables showed no significant braking distance. With ABS enabled, the average maximum deceleration achieved was 0.79 g across all conditions and speeds; it was 0.76 g for ABS disabled. If the pavement conditions were wet, the average maximum deceleration across all conditions and speeds was 0.66 g. Under dry conditions, the average maximum deceleration was 0.88 g.

Figure C-9 shows the disparity between the nine drivers. It may seem that test subject E2 got the maximum deceleration out of the car under all conditions (Mean *Max* G_x = 0.89 g); however, as mentioned previously, E2 did not perform any wet pavement maneuvers at 70 mph. Test subject T1 was far more conservative than the other drivers with an average *Max* G_x of 0.68 g.

Equivalent Constant Deceleration. The reduced data set for the nine test subjects afforded an opportunity to conduct a special analysis of differences among subjects and conditions for an equivalent constant deceleration on each run. The quantities of actual velocity V and braking distance BD are used in the following simple equation to find the equivalent constant deceleration that would account for such a braking distance from the velocity at onset:

$$a = \frac{(1.47V)^2}{2BD} \qquad [4]$$

Inputting the velocity and braking distance, solving for a, and then dividing by 32.2 ft/sec^2 gives the equivalent constant deceleration for use as a dependent variable in the following analysis.

Since neither *Stopping Condition* nor *Geometry* were significant factors in the previous analysis, they were dropped from this analysis, leaving only *ABS*, *Pavement*, and *Subjects* as independent variables. The 70 mph data were not included in this analysis because the major interest was the 40 and 55 mph data, which were both possible test speeds for subsequent studies.

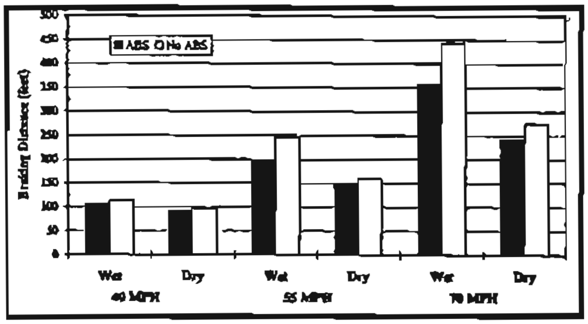

FIGURE C-7. Braking Distances for Wet and Dry Pavement Braking Maneuvers at Test Speeds of 40, 55, and 70 Miles Per Hour.

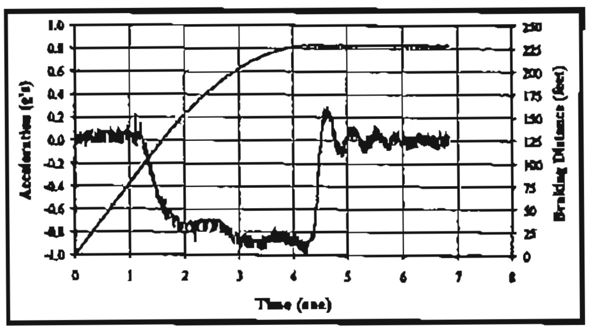

FIGURE C-8. Typical Deceleration Profile During Braking Maneuver.

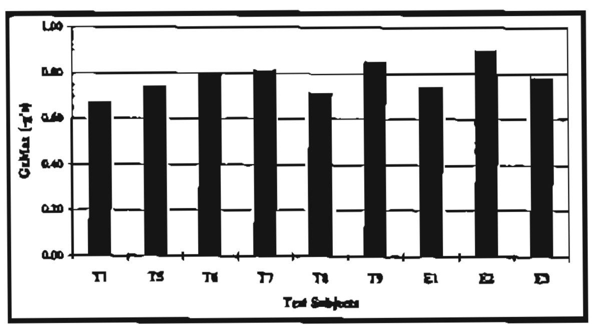

FIGURE C-9. Mean G_x Deceleration Rates Achieved by Study 1 Test Subjects.

At 40 mph, *ABS* is not a significant variable when only equivalent constant deceleration is considered. Wet or dry pavement conditions, however, are statistically significant for both the 40 and 55 mph nominal speeds. As usual, there were also statistically significant differences among drivers. Individual drivers ranged in mean equivalent constant deceleration from 0.21 to 0.31 g at 40 mph, and between 0.28 and 0.39 g at 55 mph.

Table C-21 is a summary of the equivalent constant deceleration findings. Nominal speed is given in the first column, ABS condition in Column 2, pavement condition in Column 3, and number of data points under these combinations of conditions are in Column 4. The mean equivalent constant decelerations are in Column 5, with the standard deviation (STD) associated with that mean in the next column. The last 3 columns provide an estimate of the proportions (based on this sample) of braking decelerations in the population represented by this sample.

For example, under wet conditions with no ABS at 55 mph, only five percent of the maneuvers will generate a equivalent constant deceleration of 0.30 g or less; 95 percent will generate 0.30 g or more (that is, greater deceleration). The equivalent constant deceleration values are plotted in Figure C-10. There are differences between wet and dry pavement equivalent constant decelerations, and although the differences between ABS and no ABS are very slight, the trends are certainly there. Since deceleration varies as the square of velocity, it may well be that differences in initial velocity from run to run are sufficient enough to obscure systematic differences attributable to either the ABS or pavement condition variable.

As a footnote to these results, a series of runs were made with the test vehicle using one of the expert drivers. The procedure was very simple: the driver was instructed to perform a series of straight-line stopping maneuvers on a wet pavement (same area as the 40 and 55 mph tests reported above) using the maximum pedal effort as possible (locked wheel if no ABS). ABS was either off or on. Table C-22 summarizes the results from these test runs, which should be compared to the performance reported in Table C-23 and Table C-24 for the Expert and TTI drivers under test conditions.

For example, under locked-wheel conditions at 40 mph with no ABS, the mean locked-wheel braking distance was 98 feet, but for driver braking distance under test conditions, it was 113 feet. At 55 mph, the difference between these two conditions were greater, 178 versus 246 feet. Table C-25 summarizes these differences between what the car can achieve versus what drivers will actually do, in terms of percentage of potential maximum braking performance that was exhibited by the test drivers. At highway speeds with or without ABS it is somewhere around 75 percent of full capability. It should be noted, however, that drivers may use more of the vehicle/pavement potential when confronted with an unexpected object in the roadway.

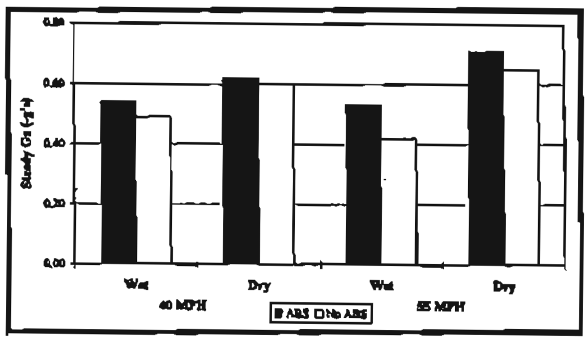

FIGURE C-10. Summary of Study 1 Mean Equivalent Constant Deceleration Values.

TABLE C-21. Summary of Equivalent Constant Deceleration (g) Percentile Values.

Speed (mph)	Pavement	ABS	N	Mean	Standard Deviation	15%	10%	5%
40	Dry	No	191	0.60	0.122	0.47	0.44	0.40
40	Dry	Yes	176	0.62	0.134	0.48	0.45	0.40
40	Wet	No	203	0.49	0.067	0.42	0.40	0.38
40	Wet	Yes	186	0.54	0.071	0.47	0.45	0.42
55	Dry	No	216	0.65	0.135	0.51	0.48	0.43
55	Dry	Yes	203	0.71	0.163	0.54	0.50	0.44
55	Wet	No	146	0.42	0.074	0.34	0.33	0.30
55	Wet	Yes	171	0.53	0.206	0.32	0.26	0.19

TABLE C-22. Summary of Locked-Wheel Vehicle Performance on a Wet Pavement.

Speed (mph)	ABS	Statistic	Braking Distance (feet)	Max G_x (g)	Equivalent Constant G_x (g)
40	On	Average	85.8	0.82	0.60
		Std. Deviation	2.3	0.04	0.02
40	Off	Average	97.8	0.77	0.56
		Std. Deviation	4.3	0.02	0.03
55	On	Average	153.6	0.82	0.59
		Std. Deviation	3.7	0.02	0.08
55	Off	Average	178.2	0.68	0.53
		Std. Deviation	8.5	0.03	0.02

TABLE C-23. Summary of Study 1 Braking Performance at 70 mph.

Variable	Condition	Braking Distance (feet)	Significant Difference	Maximum G_x (g)	Significant Difference
Condition	Surprise	316.3	No	0.78	No
	Planned	321.4		0.78	
Geometry	Tangent	309.1	No	0.77	No
	Left	339.3		0.73	
	Right	309.3		0.81	
ABS	No	347.0	Yes	0.72	No
	Yes	288.9		0.83	
Pavement	Wet	402.1	No	0.56	Yes
	Dry	259.0		0.92	
Subjects	E1	324.4		0.75	
	E2	240.5	*	0.98	*
	E3	343.5		0.72	

* *No Wet Pavement Data*

TABLE C-24. Summary of Study 1 Mean Braking Distances.

Speed (mph)	ABS	Pavement Condition	Number of Observations	Braking Distance (feet)	Geometry*	Significant Difference
40	On	Dry	177	92.3	T=98.4 L=85.8 R=91.8	Yes
40	On	Wet	188	104.8	T=106.5 L=104.8 R=102.9	No
40	Off	Dry	192	96.0	T=101.3 L=94.7 R=91.6	No
40	Off	Wet	203	112.6	T=112.5 L=112.1 R=113.2	No
55	On	Dry	200	149.4	T=140.5 L=148.8 R=156.5	No
55	On	Wet	171	199.1	T=200.7 L=196.3 R=200.2	No
55	Off	Dry	214	159.0	T=155.8 L=165.0 R=157.0	Yes
55	Off	Wet	145	246.2	T=245.5 L=245.1 R=247.9	No
70	On	Dry	64	242.8	T=234.6 L=244.8 R=247.7	Yes
70	On	Wet	43	357.4	T=345.4 L=365.5 R=359.8	Yes
70	Off	Dry	63	275.4	T=263.1 L=305.9 R=264.8	No
70	Off	Wet	47	443.0	T=440.9 L=444.0 R=443.7	No

* *T = Tangent, L = Left Curve, and R = Right Curve.*

TABLE C-25. Comparison of Maximum Versus Test Driver Braking on Wet Pavements.

	No ABS			ABS		
	Braking Distance (feet)			Braking Distance (ft)		
Speed (mph)	Maximum	Test Driver	M/T	Maximum	Test Driver	M/T
40	98	113	87%	86	105	82%
55	178	246	75%	153	199	77%

Summary of Study 1 Results

- Based on these data obtained under closed-course test conditions, with drivers prepared to brake at some time during most runs, nearly all drivers were capable of a shorter perception-brake reaction time than AASHTO's 2.5 seconds. The remaining studies focused upon PBRTvaluve when conditions were not so predictable.

- There was no need to test two types of scenarios to the onset of the red LED, *anticipated* and *surprise*. Braking performance and peak decelerations are virtually identical in either event strategy. Because *surprise* is more closely associated with real-world stopping situations, that procedure was used for all but a few practice runs in the remaining studies.

- Although a few combinations of conditions led to statistically significant differences between tangent and curved sections of test track, the majority of runs seemed to reveal no significant difference between braking on these different geometric conditions. The remaining studies included horizontal curves and tangent sections, but systematic variation of tangent, left, and right curves was not continued.

- The large differences that existed among conditions run at 55 to 40 mph, and the existence of the national 55 mile per hour speed limit, suggested that additional testing with volunteer test subjects should be done at 55 mph.

- As expected, ABS made a difference, both in braking distance and in maximum deceleration, indicating a need for additional study. It should be noted that the 1991 Chevrolet Caprice braking system was designed to have the ABS operational, and performance with that system disabled might not be exactly the same as a car not designed to have ABS; however, because any differences were probably negligible and every vehicle must be able to meet FMVSS 105 with or without ABS, it was decided to proceed with the Caprice as a test vehicle.

- The condition of the pavement resulted in significant differences in braking distance, as well as maximum longitudinal deceleration, but, perhaps surprisingly, not in an equivalent constant deceleration. Both wet and dry pavement conditions were included in the remaining studies.

- There were large individual differences in the level of performance in both braking distance and the maximum acceptable longitudinal deceleration among the drivers. Overall, drivers generated anywhere from about 0.7 to 0.9 g. An average value among the drivers was 0.78 g with a standard deviation of 0.07 g at 40 or 55 mph and 0.82 g with a standard deviation of 0.12 g for 70 mph.

- Equivalent constant deceleration also varied widely among drivers, ranging from 0.46 to 0.70 g depending upon velocity on initiating the braking maneuver. A mean value for all drivers at 40 mph was 0.56 g with a standard deviation of 0.11 g; at 55 mph, the values were 0.60 and 0.19 g, respectively. Based on the 55 mile per hour data, 85 percent of all drivers will produce equivalent constant decelerations of at least 0.34 g under wet conditions without ABS, and least 0.54 g under dry conditions with ABS. Additionally, 95 percent of all drivers will produce equivalent constant decelerations of at least 0.30 g under wet conditions without ABS, and at least 0.44 g on dry pavements with ABS.

RESULTS - STUDIES 2, 3, AND 4

The results of Studies 2, 3, and 4 are presented in this Section. Studies 2 and 3 were conducted at the Texas A&M University Riverside Campus on closed-course conditions. These two studies evaluated driver braking characteristics, namely perception-brake reaction times, braking distances, and deceleration characteristics. Study 4 was an open-road study and was conducted on a low-volume, road near the Riverside Campus. It is the intent of this study to evaluate driver's perception-brake reaction times to an unexpected object in the roadway.

This section is separated into three major sections. The first section presents the findings of the driver's perception-brake reaction times to unexpected and expected object conditions. The second section presents the results of the driver's braking distance performance to object conditions. The third section presents the driver's deceleration characteristics.

Perception-Brake Reaction Times

The results of the perception-brake reaction time tests for Studies 2, 3, and 4 are presented in this section. The perception-brake reaction time value for each test run was obtained by taking the difference in time between the onset of the unexpected barrier or the LED indicator and the instant the brake pedal was pressed by the test subject. The analysis of the data involved determining the average and upper-percentile values for perception-brake reaction time and performing ANOVA and Duncan's multiple range tests for determining significant differences due to age, gender, and type of study. The analysis was performed using the Statistical Analysis System, or SAS (*34*).

A delay factor of 0.27 seconds was subtracted from the perception-brake reaction time values obtained from the unexpected barrier segments of Studies 2 and 3. This delay factor was calculated by videotaping five simulated test runs and then analyzing the video on a frame-by-frame basis. The delay factor accounts for the time delay between the test administrator pressing the button to release the barrier to the time movement of the barrier could be detected from the driver's position on the course.

A delay factor was also subtracted from the perception-brake reaction time values obtained in Study 4, the open-road study. It was determined, also from video frames, that the time delay between the test administrator pressing the pendant button to release the barrel to the time movement of the barrel could be detected from the driver's position on the course was 0.50 seconds.

Perception-Brake Reaction Time Performance to an Unexpected Object. Of the 26 test subjects that participated in Study 2, 22 of them resulted in perception-brake reaction time events for subsequent analysis. One of the test subjects, on the approach to the unexpected barrier, pressed the brake pedal prematurely, possibly to decrease the speed; thus eliminating this value from future use. The remaining three subjects did not respond to the unexpected barrier, either driving through it or braking belatedly. For one of these three subjects and for five others that did perform a braking maneuver in response to the object, the instrumentation failed; however, even though the instrumentation failed for these five subjects, the pressure-sensitive devices on the throttle and brake still recorded the subject's perception-brake reaction time, but not a braking distance measurement.

Of the 12 test subjects in Study 3, 10 of them resulted in perception-brake reaction time events for subsequent analysis. Two of the test subjects did not react at all to the barrier, either driving through it or braking belatedly. One of the test subjects veered completely off of the course to avoid the barricade. This subject struck the barrier mounts and sustained minor damage to the vehicle. This behavior rendered the braking data useless, but still provided a usable perception-brake reaction time value for the unexpected barrier.

Of the 12 test subjects in Study 4, all but one subject provided perception-brake reaction time events for subsequent analysis. This subject did not react to the sudden appearance of the barrel from the side of the road. The others, however, did respond either by momentarily pressing the brake or steering into the opposite lane, or both, to avoid the barrel. Table C-26 summarizes the perception-brake reaction time values to an unexpected object obtained for Studies 2, 3, and 4.

The analysis of variance results indicated that there were statistically significant differences, at a 95 percent level of confidence, due to the type of study conducted. After a further analysis using Duncan's multiple range test, it was determined that the perception-brake reaction time values from Study 2, in which test subjects drove the TTI vehicle, were significantly different than the values obtained in Studies 3 and 4, where test subjects drove their own vehicle. Subjects in their own vehicles exhibited slightly longer perception-brake reaction times than subjects driving the TTI contract vehicle.

Upper-percentile perception-brake reaction time values were also calculated. The test subjects in Study 3, in their own vehicles, exhibited the longest perception-brake reaction values. A value of approximately 1.98 seconds, based on a 95 percent level of confidence, includes almost, if not all, of the drivers in these three studies. Table C-27 summarizes the upper-percentile perception-brake reaction time values.

TABLE C-26. Summary of Perception-Brake Reaction Time to an Unexpected Object.

Study	Age	No. of Test Subjects	Mean PBRT (seconds)	Standard Deviation (seconds)
Study 2	Older	12	0.82	0.159
	Younger	10	0.82	0.203
Study 3	Older	7	1.14	0.353
	Younger	3	0.93	0.191
Study 4	Older	5	1.06	0.222
	Younger	6	1.14	0.204

TABLE C-27. Percentile Estimates of Perception-Brake Reaction Time to an Unexpected Object.

Percentile	Perception-Response Time (seconds)		
	Study 2 TTI Vehicle	Study 3 Personal Vehicle	Study 4 Personal Vehicle
Mean	0.82	1.09	1.11
75th	1.02	1.54	1.40
90th	1.15	1.81	1.57
95th	1.23	1.98	1.68
99th	1.39	2.31	1.90
AASHTO	2.50	2.50	2.50

Perception-Brake Reaction Time Performance to an Expected Object. The analysis of driver's perception-brake reaction times to the expected object scenarios, or the onset of the LED indicator, was limited to Part B of Studies 2 and 3. Following the unexpected object encounter in Studies 2 and 3, each test subject performed a series of 20 braking maneuvers on a combination of dry and wet pavements and tangent and horizontal curve sections. The braking maneuver began when the test subject reacted to the LED mounted on the windshield. This LED was the *expected* object; the test subjects knew that the LED would be initiated, but were not warned or told when it would be initiated. Every subject, except one driver in Study 3, resulted in usable values for the analysis. The driver that did not provide data chose not to participate in Parts B and C of the study after veering off of the course and sustaining minor vehicle damage in Part A.

On a few of the test runs, some of the test subjects exhibited a belated reaction to the LED. An analysis of the distribution of the perception-brake reaction time data revealed approximately 95 percent of the total sample size fell below 2.25 seconds. If values larger than 2.25 seconds were included, means and standard deviations only increased, which was not representative of perception-brake reaction time data. Due to the delayed driver response not representative of an anticipated perception-brake reaction time process, perception-brake reaction time values that exceeded 2.25 seconds were considered outliers and eliminated from the data set.

Also, some of the subjects during the course of the testing anticipated the onset of the LED. The premature movement of the foot off of the brake pedal prior to the onset of the LED was sensed by the instrumentation. In these cases, the subjects exhibited unusually fast perception-brake reaction times, in the 0.1 to 0.2 second range. Therefore, perception-brake reaction time values were excluded from the data set in cases where the driver removed the foot off of the brake pedal prior to the LED signal and/or when perception-brake reaction time values did not exceed 0.25 seconds. A complete summary of the PBRT values, including a segregation by age and gender characteristics, is presented in Table C-28.

An analysis of variance for this data showed significant differences due to age and gender, with significant braking distances at 0.0001 and 0.0495, respectively. As expected, the younger subjects exhibited faster perception-brake reaction time performances than the older ones. Also, with younger and older subjects combined, the male drivers exhibited faster PBRT performance than the female drivers. The largest differences, however, remained the differences due to age. A summary of the upper-percentile perception-brake reaction time values for the two age groups is presented in Table C-29.

The analysis also demonstrated a difference in perception-brake reaction time due to the type of roadway geometry and type of pavement condition tested, both at a 0.0001 significance difference in braking distance. Subjects performed the perception-brake reaction time process faster on tangent sections than on horizontal curve sections; and performed faster during wet pavement conditions than on dry conditions. The differences between geometry conditions could be due in part to a higher driver workload required on the horizontal curve sections. The subjects were not overly familiar with the course, and driving at 55 mph through a curve between a 12-foot lane could, at best, cause momentary delays in the perception-brake reaction time process. Furthermore, it is believed that because the watering system was in operation during the course of the wet pavement braking maneuvers, the test subjects anticipated the LED signal on the shorter, wet sections of the test course, thus shortening the perception-brake reaction time process.

A summary of the PBRT values to an expected object are presented in Table C-30. An overall summary of the upper-percentile perception-brake reaction time values was determined for Studies 2 and 3, with age and gender combined, and are presented in Table C-31. These values are further illustrated in Figure C-11.

Perception-Brake Reaction Time Correction Factor. Two different types of perception-brake reaction time values have been presented thus far. One type is a driver's perception-brake reaction time to an unexpected object, or a true surprise perception and response performance. The second type is a measure of a driver's PBRT to an anticipated condition, such as in Part B of the first two studies, where the subjects had to react to the onset of the LED signal. A study described earlier (*12*) discussed an empirical correction factor to compare the differences between an *anticipated* to perception-brake reaction time a *surprise* perception-brake reaction time. Johansson and Rumar calculated an empirical correction factor of 1.35.

TABLE C-28. Summary of Perception-Brake Reaction Time to an Expected Object.

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

TABLE C-29. Percentile Estimates of Perception-Brake Reaction Time to an Expected Object.

	Perception-Brake Reaction Time (seconds)	
Percentile	Older Subjects	Younger Subjects
Mean	0.68	0.52
75th	1.02	0.70
90th	1.25	0.81
95th	1.39	0.88
99th	1.66	1.02
AASHTO	2.50	2.50

TABLE C-30. Summary of Perception-Brake Reaction Time to an Expected Object.

Pavement	Geometry	Total No. Repetitions	Mean PBRT (seconds)	Standard Deviation (seconds)
Dry	Curve	172	0.69	0.275
	Tangent	182	0.60	0.183
Wet	Curve	174	0.58	0.209
	Tangent	167	0.53	0.171

TABLE C-31. Percentile Estimates Of Perception-Brake Reaction Time to an Expected Object For Studies 2 And 3.

	Perception-Brake Reaction Time (seconds)	
Percentile	Study 2 TTI Vehicle	Study 3 Personal Vehicle
Mean	0.60	0.62
75th	0.81	0.99
90th	0.96	1.22
95th	1.05	1.36
99th	1.22	1.63

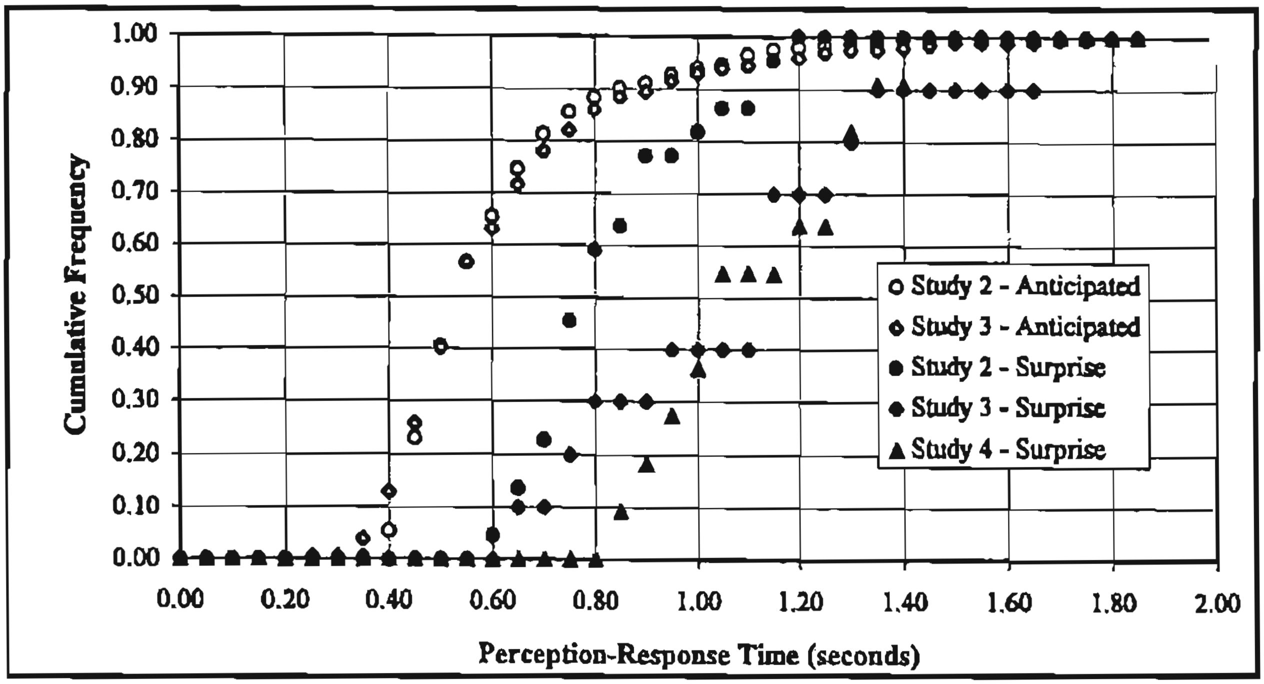

FIGURE C-11. Cumulative Frequencies of Perception-Brake Reaction Time Values.

TABLE C-32. Percentile Estimates of a Perception-Brake Reaction Time Correction Factor for Studies 2, 3, and 4.

Percentile	Perception-Brake Reaction Time (seconds)		
	Study 2 TTI Vehicle	Study 3 Personal Vehicle	Study 4 Personal Vehicle
Mean	1.38	1.77	1.80
75th	1.26	1.56	1.41
90th	1.20	1.48	1.29
95th	1.17	1.46	1.24
99th	1.14	1.42	1.17

The correction factor was based on median perception-brake reaction time values. Similarly, an empirical correction factor can be calculated for a range of upper-percentiles from the data previously presented, using the following equation:

$$\text{Correction Factor} = \frac{\text{Surprise PBRT}}{\text{Anticipated PBRT}} \qquad [5]$$

Table C-32 presents these empirical *surprise/anticipated* correction factors. For Study 2, the median correction factor is 1.38, similar to the 1.35 factor calculated by Johansson and Rumar (*12*). For Study 3, however, the median value is 1.77, which may indicate that drivers in their own vehicles may be more relaxed and comfortable, thus exhibiting longer perception-brake reaction time values. The drivers in Study 4 only performed to an unexpected object scenario. Thus, the correction values shown for Study 4 in Table C-32 are based on the *surprise* perception-brake reaction times from Study 4 and the *anticipated* perception-brake reaction times from Study 3. The median correction factor of 1.80 for Study 4 is close to the median value of 1.77 from Study 3.

Simple Reaction Time Test. In addition to the perception-brake reaction time data obtained from the *surprise* and *anticipated* braking maneuvers, test subjects in Studies 2 and 3 also provided data for a baseline perception-brake reaction time. Each subject sat behind the wheel of the stationary test vehicle, either the TTI vehicle or the personal vehicle. With their foot on the accelerator pedal, they watched for the onset of the LED signal and initiated a brake response as fast as possible at the onset of the LED. A total of ten repetitions was conducted for each test subject.

The instrumentation package was also installed in a similar car (1991 Ford Crown Victoria) as the one used in Studies 2 and 3. A series of runs identical to those described above were made under stationary conditions for a control group of volunteer test subjects from a local church. These real-world volunteer drivers, who have never before participated in empirical research studies, were all over 55 years of age, and otherwise not screened for eligibility in any way.

Table C-33 summarizes the findings from these baseline perception-brake reaction time studies. An analysis of variance indicated that gender and age differences were significant among the three studies. Males reacted faster than females, and younger drivers reacted faster than older drivers. The weak significant differences in braking distance among the three groups, those in the Caprice, those in their own cars, and those in the control group, suggests that the sample of older subjects used in the driving maneuvers are representative of the older drivers in this locale with regard to perception-brake reaction time.

Summary of Perception-Brake Reaction Time Results. Several key findings are evident from the results of the perception-brake reaction time observations from Studies 2, 3, and 4. A perception-brake reaction time value of approximately 1.98 seconds seems to be inclusive of nearly all of the subjects' performances to the unexpected obstacle that suddenly appeared in the road. Most of the test subjects responded to this true surprise condition by braking the vehicle, while a few, such as in Study 4, steered to avoid the object because the opposite lane was open for this type of maneuver.

The test subjects responded quicker while driving someone else's vehicle; in this case, the TTI contract vehicle. Perception-brake reaction time values for the subjects in Study 2 were approximately 75 percent of the perception-brake reaction time values of the test subjects in their own vehicles. Furthermore, from a Duncan's multiple range test, there did not appear to be any significant difference between the *surprise* perception-brake reaction time values for subjects in their own vehicles performing on a closed course versus performing on an open-road test environment. The mean PBRT for a test subject in this study, in his or her own vehicle, to a truly unexpected object was 1.1 seconds.

On average, the test subjects in this study exhibited an anticipated perception-brake reaction time of approximately 0.55 seconds. Significant differences were noted between age groups and gender. The younger subjects averaged 0.52 seconds and the older subjects averaged 0.68 seconds. Furthermore, the male subjects exhibited an average anticipated perception-brake reaction time of 0.59 seconds and the female subjects exhibited an average anticipated perception-brake reaction time of 0.63 seconds. Significant differences were also found between pavement conditions and geometry conditions. On the average, the test subjects responded quicker for the wet pavement braking maneuvers, with the likely explanation being that they knew the maneuver would be performed on the wet pavement section and were anticipating the LED signal. The subjects also responded faster on the tangent sections versus the horizontal curve sections.

The baseline reaction times of the older test subjects sampled from the control group did not significantly differ from those of the older test subjects that participated in Studies 2 and 3. As expected, significant differences were noted between age and gender groups, but no other significance existed among the three subject groups. From the analysis, it appears that the older subjects tested in Studies 2 and 3 are representative of other older drivers in the community and did not introduce any biases in the study results.

A *surprise* versus *anticipated* perception-brake reaction time correction factor was calculated from the available perception-brake reaction time data. A median value obtained in Study 2 was 1.38, where the test subjects were driving a vehicle other than their own. For Studies 3 and 4, the median values were slightly higher—1.77 and 1.80, respectively. The factor from Study 4, however, may not be an accurate assessment of the differences because the test subjects in Study 4 were not tested for any *anticipated* braking maneuvers. The *anticipated* perception-brake reaction time values from Study 3 were applied to the *surprise*perception-brake reaction time values from Study 4 to obtain this correction factor. A more accurate assessment is the Study 3 correction factor of 1.77.

TABLE C-33. Results of Baseline Perception-Response Time Observations.

Study	Age	Gender	No. of Test Subjects	Total No. Repetitions	Mean PBRT (seconds)	Standard Deviation (seconds)
Study 2	Older	Female	7	70	0.50	0.09
		Male	7	71	0.47	0.06
	Younger	Female	6	60	0.47	0.13
		Male	6	60	0.42	0.08
Study 3	Older	Female	5	50	0.47	0.08
		Male	3	30	0.44	0.10
	Younger	Female	2	20	0.39	0.06
		Male	1	10	0.48	0.04
Church	Older	Female	10	94	0.52	0.19
		Male	8	81	0.43	0.14

A likely explanation for the median value in Study 3 being larger than the median value in Study 2 may be that test subjects were more alert in the TTI contract vehicle during the unexpected object scenarios, causing them to respond quicker to the unexpected obstacle. One could argue, however, that drivers in their own vehicles are familiar with the foot pedal arrangements, the instrumentation panel, and other interior features of their own cars, and would respond quicker than they would driving a car with unfamiliar features. The opposite, however, appears to be the case in this research. The test subjects were likely more comfortable and relaxed in their own vehicle, thus exhibiting slightly slower responses to the unexpected obstacle. The higher values for Study 3 may be representative of how drivers perform to a truly unexpected object on the roadway, since the unexpected object scenario, even under safe and protected conditions, closely represented what a driver might encounter on the roadway—a completely unexpected event.

Braking Distances

The braking distance value, not including the distance traveled during the perception-brake reaction time, was obtained by taking the difference in distance recordings between the time the brake pedal was pressed and the time the vehicle came to a complete stop. The stopping point was easily identifiable because the instrumentation recorded a constant distance, signifying no translation on the fifth-wheel device.

The analysis of the braking data involved determining the average and upper-percentile (75th, 90th, 95th, and 99th) values and performing ANOVA tests for determining significant differences due to the type of study (test vehicle used), condition of the study (*surprise* versus *anticipated*), ABS enabled or disabled, pavement conditions, and geometry conditions. The analysis was again performed using SAS (*34*).

The data analysis revealed braking distance values that exceeded 400 feet and values less than 100 feet. For these cases, the test subject either failed to respond or responded late to the LED signal or to the unexpected object scenario, the instrumentation failed, or a combination of both. Therefore, the braking distance values in the data set greater than 400 feet or less than 100 feet were considered as non-representative behavior and were eliminated from the data analysis.

Furthermore, the results that are presented here are based upon the *nominal* rather than the actual velocity of the vehicle at the onset of the braking maneuver. The nominal velocity, with respect to the measured braking distance, was 55 mph. Braking distances where the initial velocity estimate exceeded 55 plus or minus three mph were considered suspect outliers and were excluded from the data analysis.

Braking Performance to an Unexpected Object. The braking distance values were analyzed from Part A of Studies 2 and 3. This part of each study provided braking distance values for a completely unexpected object, much like the assumptions made in the current AASHTO stopping sight distance model. This part of the study provided a unique opportunity to not only test vehicle performance characteristics, but also driver performance to a complete surprise.

Of the 26 test subjects that participated in Study 2, only 13 resulted in braking distance data for the analysis. One of the subjects pressed the brake pedal prematurely, three of the subjects did not respond to the barrier, the instrumentation failed for five others, and four of the subject's speeds were not within the desired range of 52 to 58 mph. Therefore, a total of 13 usable data values were included for the analysis. For the same reasons, only 7 of the 12 test subjects in Study 3 provided usable data for the analysis.

As mentioned several of the test subjects in both studies did not brake in response to the unexpected barrier that appeared before them. They either drove through the barricade without braking or braked momentarily, only then to realize that they could not stop before striking the obstacle. One likely explanation for this reaction is that their participation in a controlled test may have led them to believe that no harm could be done to them by driving as instructed. As mentioned earlier, one test subject veered completely off of the course, striking the barrier mounts and sustaining minor damage to the vehicle. These data were excluded from the analysis.

A summary of the braking distances to the unexpected barrier is presented in Table C-34. As previously mentioned, all of the maneuvers were performed on a dry tangent section of the course. The test vehicle in Study 2 either had the ABS processor enabled or disabled, and none of the test vehicles in Study 3 were equipped with ABS.

Note that there are large standard deviations associated with the braking distances. The large standard deviations may be consistent with the varying responses to a completely unexpected obstacle. For some of the test subjects, the obstacle appeared dangerous and the driver's rationale was to stop as quickly as possible. For others, the obstacle did not appear as dangerous, but still caused them to initiate a braking maneuver.

From the results listed in Table C-34, there were no obvious improvements in braking distance performance with ABS enabled. Many of the subjects did not completely "stomp" on the brakes during the entire braking maneuver, but only toward the end of the maneuver when it was evident that the obstacle would be struck. In other words, the closer to the obstacle, the higher the tendency the subject had to apply full pedal pressure. If the test subjects would have chosen to apply full pedal pressure, or would have been instructed to do this prior to the testing, improved braking performance with ABS may have been evident.

Braking Performance to an Expected Object. The analysis of subject's braking performance to the expected object scenarios, or the onset of the LED indicator, was limited to Part B of Studies 2 and 3. Following the unexpected object encounter in Studies 2 and 3, each test subject performed a series of 20 braking maneuvers on a combination of dry and wet pavements and tangent and horizontal curve sections. The braking maneuver began when the test subject reacted to the LED signal mounted on the windshield. Every test subject, except one driver in Study 3, resulted in usable braking data for the analysis. The driver that did not provide data chose not to participate in Parts B and C of the study after veering off of the course and sustaining minor damage to the vehicle in Part A.

As with the other studies, outliers or suspect data points were eliminated from the analysis. On a few braking maneuvers, the subjects did not respond immediately to the LED, and actually performed the braking maneuver on two geometric test sections. These values were eliminated. Also, because of the importance placed on the instructions to the driver to stay within the 12-foot lane at all times, braking maneuvers that were outside of the lane or test course were excluded. Braking maneuvers outside of the lane occurred more frequently on the wet pavement and on horizontal curves and more often on only the first or second maneuver for a particular test subject. Once the subject realized the capabilities, or limitations, of the vehicle, vehicle control improved for subsequent test runs. Test subjects performing with ABS enabled in Study 2 did not experience this problem. A complete summary of the braking performance is presented in Table C-35.

A further comparison was made with the AASHTO design value of 336 feet, the derived braking distance for a design speed of 55 mph. For these studies, test subjects in their own vehicle (all without ABS) exhibited a 90th percentile braking distance of 330 feet on a wet tangent and 342 feet on a wet curve. Test subjects in the TTI contract vehicle exhibited a 90th percentile braking distance of 244 feet on a wet tangent and 278 feet on a wet curve without ABS. The upper-percentile braking distance values for maneuvers on a wet pavement without ABS are presented in Table C-36.

It appears that the AASHTO braking distance is extremely conservative and for these studies, is inclusive of nearly all tangent braking maneuvers on a wet pavement. For both studies, the estimates for the wet curve maneuvers were longer than for the tangent maneuvers. During the testing, if the wheels locked on a wet horizontal curve, the driver usually lost control of the vehicle. If loss of control of the vehicle occurred, the subject was instructed to take whatever measures necessary in the next braking maneuver to remain in the designated lane. The subjects did so, but at the expense of slightly longer braking distances.

TABLE C-34. Summary of Braking Distance to an Unexpected Object.

Study	ABS	No. of Test Subjects	Mean Braking Distance* (feet)	Standard Deviations (feet)
Study 2	No	6	159	19
	Yes	7	161	25
Study 3	No	7	184	38

Initial Speed Equals 55±3 mph

TABLE C-35. Summary of Braking Distance to an Expected Object.

Study	ABS	Pavement	Geometry	Total No. Repetitions	Mean Braking Distance (feet)	Standard Deviation (feet)
Study 2	No	Dry	Curve	62	198	37
			Tangent	54	201	31
		Wet	Curve	54	225	29
			Tangent	49	204	22
	Yes	Dry	Curve	48	195	47
			Tangent	40	193	44
		Wet	Curve	50	199	39
			Tangent	49	185	31
Study 3	No	Dry	Curve	38	201	50
			Tangent	38	203	54
		Wet	Curve	50	199	39
			Tangent	49	185	31

TABLE C-36. Percentile Estimates of Braking Distance to an Expected Object.

	Braking Distance on Wet Pavement, No ABS (feet)			
	Study 2 - TTI Vehicle		Study 3 - Personal Vehicle	
Percentile	Tangent	Curve	Tangent	Curve
Mean	204	225	232	273
75th	228	257	293	302
90th	244	278	330	342
95th	254	291	353	366
99th	273	316	398	414
AASHTO*	336	336	336	336

* *Braking Distance at 55mph*

Summary of Braking Distance Results. An analysis of variance of the combined data sets from the unexpected object segment with the expected object segment showed statistically significant differences due to the vehicle driven, pavement conditions, and ABS, with further significance amongst the interactions of *Study and Pavement* and *Study, ABS and Pavement* . The variable *Study* in this analysis indicates the data from either Study 2 or Study 3.

Effects of Study Type – Anticipated Versus Surprise. The test subjects, on the whole, did not perform as well (i.e., they had longer braking distances) to the *anticipated* conditions as they did to the complete *surprise* condition; an analysis of variance of the data showed a significant difference ($p = 0.0038$). Drivers exhibited marginally improved braking distances, on the order of approximately 25 percent, to the surprise object, probably because this scenario appeared more hazardous and they were willing to decelerate at a higher rate. The differences are presented in Figure C-12.

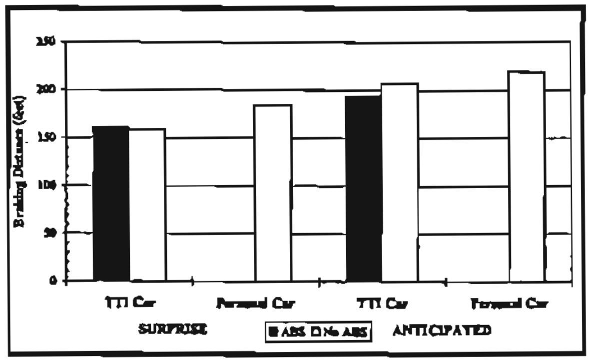

FIGURE C-12. Average Braking Distance Values for Surprise and Anticipated Object in Studies 2 and 3.

Effects of Geometry – Tangent Versus Curve. Surprisingly, there were no significant differences in the braking distances due to geometry conditions. As illustrated by the results presented in Figure C-13, the subjects performed about as well on the horizontal curves as they did on the tangent sections. One possible explanation for this result is that the test subjects were not utilizing the full frictional capabilities of the pavement or the full braking ability of the test vehicle, especially on the tangent sections. The same result was demonstrated in Study 2.

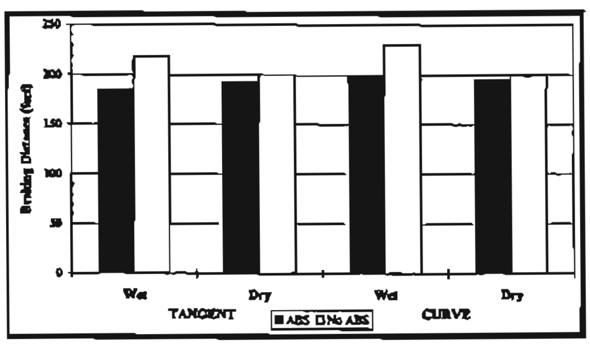

FIGURE C-13. Average Braking Distance Values for Anticipated Object in Studies 2 and 3.

Effects of Antilock Brakes – Enabled Versus Disabled. Marginal improvements in braking distance, on the order of approximately 10 to 15 percent, were demonstrated with the use of antilock brakes. The improvements are primarily demonstrated by braking maneuvers on a wet pavement and on horizontal curves. At 55 mph on a wet horizontal curve, the test subjects operating the TTI contract vehicle with ABS managed to stop the vehicle, with control, in approximately 200 feet. Subjects operating the same vehicle with the ABS processor disabled, or in their own personal vehicles, required approximately 225 feet and 237 feet, respectively, to stop, with a lesser degree of vehicle control.

Decelerations

The results of the vehicle acceleration or deceleration characteristics during the braking maneuver from Studies 2 and 3 are presented in this section. The deceleration value of interest for this study was the maximum longitudinal deceleration achieved during a particular braking maneuver, referred to as *Max* G_x. This value is not sustained throughout the braking maneuver, but achieved at some time during the braking maneuver. The instrumentation also recorded lateral accelerations, but these data were not analyzed for this study.

The analysis of the deceleration data involved determining the average and upper-percentile values and performing ANOVA tests for determining significant differences due to the type of study (test vehicle used), condition of the study (*surprise* versus *anticipated*), ABS enabled or disabled, pavement conditions, and geometry conditions.

The same data set used to analyze driver braking distances in the previous section of this study was analyzed for the driver deceleration data. Deceleration data indicating velocity estimates in excess of 55 plus or minus three mph were considered outliers and were excluded from the data analysis. Other data points excluded from the analysis were the test runs where the test subject failed to respond to the object scenario, when the instrumentation failed, or when the test subject performed the braking maneuver outside of the lane, or on two different geometric sections.

Except for the excluded data, every braking maneuver provided a *Max* G_x value, even for a few runs in which the accelerometer package experienced a minor malfunction. On a few of the initial braking maneuvers in Study 2, acceleration solenoids in the accelerometer package recorded excessive fluctuations in the g-forces of the vehicle, exhibiting *Max* G_x values exceeding 1.25 g. An estimate of *Max* G_x, however, could still be assessed after plotting the time history of the longitudinal deceleration and identifying an average between the fluctuations. The initial maneuvers in which the malfunction occurred were not excluded from the data analysis.

Deceleration Performance to an Unexpected Object, Maximum Deceleration. The deceleration values were analyzed from the braking maneuvers performed in Part A of Studies 2 and 3. The data set was the same one analyzed in the previous section for driver braking distances. The test runs that did not qualify for the analysis of vehicle braking distances were also excluded from this analysis. Similarly, the same test subjects that provided usable data for the braking distance analysis provided deceleration values for this analysis. In summary, 13 of the 26 test subjects in Study 2 and 7 of the 12 test subjects in Study 3 resulted in usable data.

A summary of the *Max* G_x deceleration characteristics to the unexpected barrier is presented in Table C-37. As previously mentioned, all of the maneuvers were performed on a dry tangent section of the course. The test vehicle in Study 2 either had the ABS processor enabled or disabled, and none of the test vehicles in Study 3 were equipped with ABS.

The mean values reported below correlate closely with the friction coefficient of the dry pavement. The peak longitudinal deceleration, however, was usually not exhibited until the last portion of the braking maneuver. As the vehicle was constantly decreasing in speed and as the driver came closer to the obstacle, the pedal pressure on the brake increased, to a point, at times, to a complete lock-up of the wheels.

The results demonstrate that test subjects in their own vehicles are not willing to subject themselves to as high deceleration forces as test subjects in someone else's vehicle. Test subjects in Study 3 exhibited an average *Max* G_x of 0.74 g. Whereas, the subjects in Study 2, operating without ABS, exhibited an average *Max* G_x of 0.91 g. The difference in *Max* G_x values between the two studies corresponds to the results presented in the braking distance section of this section, which indicated that test subjects in Study 2, without ABS in the TTI contract vehicle, exhibited shorter braking distances than the test subjects in their own vehicles in Study 3.

TABLE C-37. Summary of *Max* G_x to an Unexpected Object.

Study	ABS	No. of Test Subjects	Mean Max Gx* (g)	Standard Deviations (g)
Study 2	No	6	0.91	0.08
	Yes	7	0.91	0.14
Study 3	No	7	0.74	0.09

* *Dry Pavement Conditions*

Equivalent *Constant* Deceleration. The analysis was taken one step further to examine one of the assumptions of the AASHTO model: the constant deceleration value sustained throughout the braking maneuver. The reduced data set provided an opportunity to derive a equivalent constant deceleration, or equivalent constant G_x, for each test run. Solving for a and dividing by 32.2 ft/sec^2 gives the equivalent constant deceleration values listed in Table C-38. Upper-percentile estimates of the data set are listed in Table C-39. The values presented in the table are, of course, a result of dry pavement braking maneuvers. If the pavement conditions were wet for the unexpected object scenario, the braking maneuver would have more closely represented an AASHTO stopping maneuver (i.e., unexpected object encounter on a wet pavement). The braking distances would be longer on a wet pavement because of the reduced pavement friction capabilities, resulting in lower estimates of equivalent constant G_x. The analysis will further investigate the equivalent constant deceleration derivation and the effects of wet pavement braking maneuvers in subsequent sections.

Deceleration Performance to an Expected Object, *Maximum Deceleration*. The analysis of driver and vehicle deceleration performance to the expected object scenarios was again limited to Part B of Studies 2 and 3. All of the test subjects in both studies, with the exception of the one subject that chose not to participate, resulted in usable deceleration data for the analysis.

The same data set used for the braking distance analysis to an expected object was again used for this portion of the analysis. Similarly, test runs where subjects unintentionally performed the braking maneuver on two geometric test sections were eliminated, as were the test runs where the vehicle went outside of the lane or off of the course during the braking maneuver.

The results indicated that the maximum peak longitudinal deceleration to an expected object scenario was lower than for an unexpected object scenario. For example, the mean *Max* G_x on a dry tangent section, for drivers in the TTI contract vehicle without ABS, was 0.63 g to the expected object scenarios. The same drivers exhibited 0.91 g to the unexpected object scenario. For the drivers in Study 3, a median *Max* G_x on a dry tangent was 0.68 g, which was slightly lower than 0.74 g for an unexpected object.

Antilock brakes also had a significant effect on the maximum peak longitudinal deceleration achieved during the expected object braking maneuvers, allowing the driver to achieve higher deceleration forces. On the wet tangent braking maneuvers, subjects in Study 2 exhibited 0.71 g with ABS. In Study 3, subjects driving their own vehicles without ABS exhibited median *Max* G_x rates of 0.63 g. A summary of the *Max* G_x deceleration characteristics to the expected object is presented in Table C-40.

Equivalent Constant Deceleration. The equivalent constant deceleration values for each test run to the expected object was obtained from the data set. These values are presented in Table C-41. A further comparison of the equivalent constant deceleration values to the current AASHTO model is shown in Table C-42. Also presented in Table C-42 are the upper-percentile equivalent constant deceleration values obtained from braking maneuvers on wet tangent sections without ABS enabled. A wet pavement braking maneuver without ABS closely represents an AASHTO braking maneuver, a maneuver that assumes a locked-wheel braking maneuver on a poor, wet pavement.

The mean equivalent constant decelerations of 0.49 g and 0.45 g for Studies 2 and 3 without ABS, respectively, was higher than anticipated. Even the peak deceleration rate (*Max* G_x) achieved during the wet pavement maneuvers was higher than anticipated, with a median *Max* G_x of 0.63 g in Studies 2 and 3. The peak value, however, was usually achieved at some point toward the end of the braking maneuver when the friction capabilities of the pavement increased as the vehicle slowed to a stop. The vehicle's tires were a factor in what level of deceleration could be achieved. Good tires can provide frictional capabilities higher than the measured skid number of the pavement, especially toward the end of the braking maneuver as a vehicle is slowed to a stop.

Deceleration Correction Factor. Similar to the procedure to obtain a perception-brake reaction time correction factor with *anticipated* and *surprise* perception-brake reaction time values, a further analysis of the deceleration data was conducted to examine the differences between anticipated and surprise deceleration characteristics. The braking maneuvers to the unexpected object were only conducted on a dry tangent, with and without ABS, depending upon the study. Therefore, in order to compare similar values for *surprise* versus *anticipated* conditions, data were only examined from the dry tangent maneuvers from Parts A and B of Studies 2 and 3, including the differences between ABS and no ABS performance. The following formula was used to obtain a range of percentile estimates of the deceleration correction factor:

$$Correction\ Factor = \frac{Surprise\ Deceleration}{Anticipated\ Deceleration} \quad [6]$$

This factor was established for the maximum peak longitudinal deceleration achieved for each of the dry tangent maneuvers (*Max* G_x) and for the equivalent constant deceleration value (*Equivalent Constant* G_x) calculated as a function of the braking distance and initial velocity prior to the onset of the braking maneuvers. The values for the *Max* G_x and *Equivalent Constant* G_x correction factors are presented in Table C-43.

The trends indicate a median *Max* G_x correction factor in the range between 1.1 and 1.3, depending upon the study. The test subjects exhibited higher peak decelerations during the *surprise* maneuvers, and it is evident with the correction factors greater than 1.0. The range indicates that the test subjects were willing to accept higher deceleration levels to a seemingly hazardous and unexpected object and to be more

TABLE C-38. Summary of Equivalent constant G_x to an Unexpected Object.

Study	ABS	No. of Test Subjects	Mean Equivalent Constant G_x* (g)	Standard Deviations (g)
Study 2	No	6	0.62	0.07
	Yes	7	0.63	0.08
Study 3	No	7	0.55	0.07

**Dry Pavement Conditions*

TABLE C-39. Percentile Estimates of Equivalent Constant G_x to an Unexpected Object.

	Equivalent constant Deceleration (g)		
	Study 2 ABS	**Study 2 No ABS**	**Study 3 No ABS**
Mean	0.63	0.62	0.55
75th	0.50	0.49	0.43
90th	0.42	0.42	0.37
95th	0.38	0.38	0.32
99th	0.28	0.29	0.24
AASHTO*	0.30	0.30	0.30

* *Assumed Wet Pavement Friction Coefficient at 55 MPH*

TABLE C-40. Summary of *Max* G_x to an Expected Object.

Study	ABS	Pavement	Geometry	Total No. Repetitions	Mean *Max* G_x (g)	Standard Deviation (g)
Study 2	No	Dry	Curve	62	0.68	0.11
			Tangent	54	0.70	0.13
		Wet	Curve	56	0.61	0.06
			Tangent	50	0.63	0.06
	Yes	Dry	Curve	48	0.73	0.18
			Tangent	40	0.76	0.18
		Wet	Curve	51	0.68	0.11
			Tangent	49	0.71	0.09
Study 3	No	Dry	Curve	38	0.66	0.14
			Tangent	38	0.68	0.14
		Wet	Curve	38	0.58	0.13
			Tangent	43	0.63	0.09

TABLE C-41. Summary of Equivalent Constant G_x to an Expected Object.

Study	ABS	Pavement	Geometry	Total No. Repetitions	Mean *Max* G_x (g)	Standard Deviation (g)
Study 2	No	Dry	Curve	62	0.54	0.20
			Tangent	54	0.53	0.08
		Wet	Curve	54	0.45	0.04
			Tangent	49	0.49	0.04
	Yes	Dry	Curve	48	0.54	0.11
			Tangent	40	0.57	0.12
		Wet	Curve	50	0.51	0.09
			Tangent	49	0.55	0.08
Study 3	No	Dry	Curve	38	0.53	0.11
			Tangent	38	0.54	0.11
		Wet	Curve	38	0.42	0.06
			Tangent	43	0.45	0.06

TABLE C-42. Percentile Estimates of Equivalent Constant G_x to an Expected Object.

	Equivalent Constant Deceleration (g)		
	Study 2 ABS	Study 2 No ABS	Study 3 No ABS
Mean	0.55	0.49	0.45
75th	0.46	0.44	0.36
90th	0.40	0.41	0.31
95th	0.37	0.39	0.27
99th	0.30	0.35	0.21
AASHTO*	0.30	0.30	0.30

**Assumed Wet Pavement Friction Coefficient at 55 mph*

TABLE C-43. Mean Estimates of an Empirical Correction Factor.

	Median Correction Factor		
Variable	Study 2 ABS	Study 2 No ABS	Study 3 No ABS
Max G_x	1.20	1.30	1.09
Equivalent Constant G_x	1.14	1.27	1.22

conservative in the *anticipated* maneuvers, perhaps because of their unwillingness to continually subject their own vehicles to repeated, harsh braking maneuvers.

The trends for the median equivalent constant G_x correction factor show a range between 1.14 to 1.27. Again, the results suggest consistently higher equivalent constant deceleration rates during the *surprise* maneuvers. Although not indicative of the values in Table C-43, the test subjects did perform at higher equivalent constant deceleration rates in the TTI contract vehicle (without ABS) than they did in their own vehicle. The higher decelerations may be for the reason mentioned previously: that they were less willing to subject their own vehicle to the harsh, repetitious braking maneuvers.

Summary of Deceleration Results. An analysis of variance of the dependent variable *Max* G_x showed statistically significant differences due to the vehicle driven, pavement conditions, ABS, geometry and whether the study conditions were *anticipated* or *surprise* maneuvers. The analysis of the dependent variable *equivalent constant* G_x also indicated significance due to the vehicle driven, pavement conditions, ABS, and geometry. Significant interactions of the independent variables Study x Pavement and Study x ABS x Pavement were also shown for the dependent variable *Equivalent Constant* G_x.

Effects of Study Type -- Anticipated Versus Surprise. The analysis showed a significant braking distance of *Max* G_x due to the conditions of the study. For the *anticipated* object maneuvers, test subjects driving the TTI contract vehicle exhibited higher levels of deceleration than the ones driving their own vehicles. These differences are shown in Figure C-14. As previously discussed, it appears that test subjects were not as willing to subject their own vehicles to higher levels of deceleration, or at least not in a testing environment, because of the risks involved. Conversely, an argument could be made that a driver should know the capabilities of their own vehicle and would, therefore, exhibit higher decelerations. The findings, however, do not support this argument.

Effects of Pavement Conditions -- Wet Versus Dry. An analysis of variance of the dependent variables *Max* G_x and *Equivalent Constant* G_x indicated that statistically significant existed due to pavement conditions at a 0.0001, which was expected. As shown in Figure C-15, the maximum peak deceleration was higher on dry pavements, which provide higher frictional qualities than the wet pavements.

Effects of Antilock Brakes -- Enabled Versus Disabled. An analysis of variance with the independent variable *ABS* also indicated statistically significant braking distance differences ($p < 0.0001$) for both *Max* G_x and *Equivalent Constant* G_x. Also shown in Figure C-15 is that the peak decelerations were higher with ABS enabled. The higher decelerations obtained with ABS support the previous discussion that not only does ABS provide improved vehicle control, but also an improved utilization of the friction capabilities of the tire/pavement.

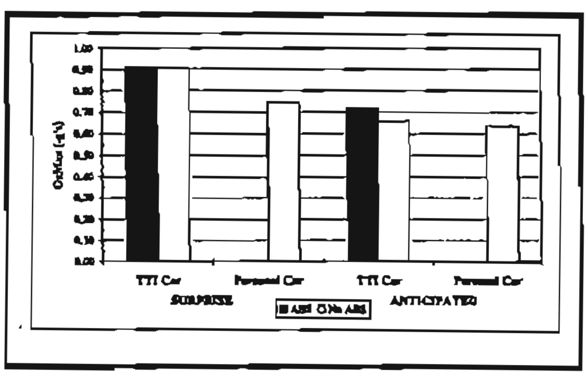

FIGURE C-14. Average *Max* G_x Values for Surprise and Anticipated Object in Studies 2 and 3.

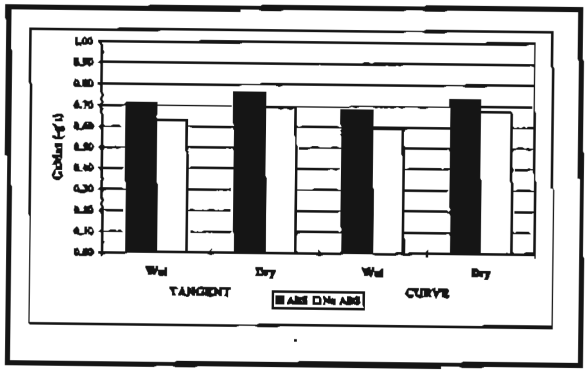

FIGURE C-15. Average *Max* G_x Values for Anticipated Object in Studies 2 and 3.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The following conclusions have been drawn from the results of the four closed- and open-road field studies described in this appendix. For the reader's convenience, they are grouped in terms of perception-brake reaction time, braking distance, maximum and equivalent constant deceleration, and other conclusions.

Perception-Brake Reaction Time

- Reaction time and foot movement time were essentially independent of one another for the same driver or across drivers; hence, simple serial chaining can be used to describe the events that make up perception-brake reaction.

- The ratio of perception-brake reaction time to a totally unexpected event, to that of an anticipated or expected event ranged from 1.35 to 1.8, depending on the study conditions. These values can be compared to Johansson and Rumar's (*12*) ratio of 1.35 for somewhat similar conditions, since the drivers in both studies were in their own cars.

- An average perception-brake reaction time to a total surprise, but controlled-road condition, is about 1.10 seconds; this lengthens marginally to 1.11 seconds under open-road conditions. If these findings are extrapolated to the 95th percentile, perception-brake reaction times would be 1.98 and 1.68 seconds, respectively. These findings are consistent with those in the literature, in that almost all drivers are capable of responding to an unexpected hazard in 2.0 seconds or less.

- AASHTO's design perception-brake reaction time of 2.5 seconds is a very conservative value that includes a 0.5 second factor of safety over and above the perception-brake reaction time requirements of most drivers. This additional time should be recognized for what it is: a safety margin to accommodate other factors, such as the needs of fatigued or impaired drivers that are capable of perceiving an object in the roadway.

Braking Distances

- When compared to conventional braking systems, ABS resulted in shorter braking distances, as much as 100 feet at 55 mph. Drivers take advantage of these systems if they are aware of their existence and experienced in their use; however, if drivers are not aware or experienced, driver braking performance tends to approach that of non-ABS braking systems.

- When compared to dry pavements, there was a significant increase in driver braking performance on wet pavements. Driver braking performance on wet pavements is enhanced by ABS in terms of better control and shorter braking distances.

- When compared to tangents, no large difference was noted in driver braking performance on horizontal curves designed to minimum AASHTO standards. Drivers tend to be less aggressive when braking on curves resulting in slightly longer braking distances. Although these differences are large enough to be of statistical significance for some combinations of conditions, they are not large enough to be of practical concern.

- Under an unexpected surprise, but a closed-road condition, an average driver braking distance at 55 mph on dry pavement is 170 feet.

Maximum and Equivalent Constant Deceleration

- Actual deceleration profiles are anything but linear, rather they resemble a step input with higher-order components. The maximum deceleration achieved is relatively independent of initial velocity, at least in the range of speeds tested (40, 55, and 70 mph).

- There were large individual differences in driver performance levels in terms of maximum longitudinal deceleration. For some drivers, maximum longitudinal deceleration was equal to the coefficient of friction of the pavement; however, the average maximum deceleration was about 75 percent of that level. Overall, drivers generated maximum decelerations from 0.7 g to 0.9 g. An average value among the drivers was 0.78 g with a standard deviation of 0.07 g at 40 or 55 mph, and 0.82 g with a standard deviation of 0.12 g for 70 mph.

- *Equivalent constant* deceleration also varied widely among drivers, ranging from 0.46 g to 0.70 g. A mean value for all drivers at 40 mph was 0.56 g with a standard deviation of 0.11 g; at 55 mph, the values were 0.60 g and 0.19 g, respectively. Based on the 55 mph data, 85 percent of all drivers will produce *equivalent constant* decelerations of at least 0.34 g under wet conditions without ABS, and the same percentage will produce "equivalent constant" decelerations of at least 0.54 g under dry conditions with ABS. Additionally, 95 of all drivers will produce at least 0.30 g under wet conditions without ABS, and at least 0.44 g on dry pavements with ABS.

- Calculated *equivalent constant* deceleration levels for drivers in both a total surprise maneuver and in response to the onset of a signal are very similar, somewhere around 0.5 g on either a wet or dry pavement. *Equivalent constant* deceleration estimates were slightly lower for wet pavements than they were for dry pavements. Such slight (but statistically significant) differences were also noted between curves and tangents.

Other Conclusions

- The condition of the pavement, wet or dry, made a rather large and statistically significant difference in driver braking distance, as well as maximum longitudinal deceleration, but, surprisingly, not in a derived *equivalent constant* deceleration.

- Drivers are slower to respond to a completely unexpected, but not threatening, obstacle in their own vehicles than they are in someone else's vehicle.

- From follow-up questions in Study 4, drivers were all surprised by the barrel rolling down the ramp from the pickup truck beside the open highway. They remarked that at highway speeds and with no oncoming traffic, braking or attempting to stop was less desirable than trying to perform an evasive maneuver.

Recommendations

The following recommendations are based on the results of the four controlled- and open-road field studies described in this appendix. For the reader's convenience, they are grouped in terms of perception-brake reaction time, braking distance, and maximum and equivalent constant deceleration.

Perception-Brake Reaction Times

- For stopping sight distance design purposes, the AASHTO estimate of 2.5 seconds for perception-brake reaction time appears to be more than adequate for most drivers. A perception-brake reaction to an unexpected object in the roadway is 1.10 seconds, and almost all drivers are capable of beginning a brake response within 1.7 to 2.0 seconds.

- Perception-brake reaction time to an unexpected hazard is from 1.35 to 1.8 times larger than that to an anticipated object. This correction factor should be used to predict performance, if one is extrapolating from a typical experimental study. The mean perception-brake reaction time to an anticipated object is 0.65 seconds, and almost all drivers are capable of responding to the anticipated object within 1.5 seconds.

Braking Distances

- Until such time that the vehicle fleet has transitioned to primarily one with ABS vehicles, braking distance estimates should be based on non-ABS brake performance. Because all vehicles must meet Federal Motor Vehicle Safety Standard 105, it should serve as the minimum vehicle performance benchmark for design purposes. Almost all vehicles are capable of very high decelerations, especially under dry pavement conditions; however, drivers may not reach those levels, with or without ABS.

- When compared with horizontal curves without superelevation that are designed to AASHTO minimum criteria, braking distances on tangents are essentially the same. Thus, straight-line braking distances should be used for design purposes.

Decelerations

- The average drivers will accept somewhere around 75 percent of maximum braking performance on a given pavement and set of physical conditions, but large individual differences exist in "comfortable" braking effort. Under wet pavement conditions, 0.3 g could be expected of all but 5 percent of drivers. An average level of deceleration on either wet or dry pavement is 0.5 g.

- Drivers do not greatly change their braking comfort level for different kinds of perceived hazards that are not an imminent threat to their safety. A situation, such as an object in the roadway, could be expected to induce a higher level of deceleration to the physical limits of the vehicle and highway.

- Average driver decelerations in response to traffic signals are less than average driver decelerations in response to an anticipated object, such as what was presented in this research. Furthermore, the anticipated decelerations are less than decelerations to completely unexpected object, which again, was presented in this research, but in closed-course conditions. It is believed that, if a completely unexpected object was encountered on an open roadway, drivers would decelerate at even higher rates than what was exhibited in the closed-course, unexpected object braking maneuvers.

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APPENDIX D -

DRIVER VISUAL CAPABILITIES

INTRODUCTION

Stopping sight distance (SSD) influences a variety of geometric design values including length and sharpness of horizontal and vertical curvature. To provide longer stopping sight distance values, horizontal curves are flattened (degree of curvature is decreased), while vertical curves are lengthened. Depending on whether minimum or desirable stopping sight distance values are used as the design criteria, the construction cost of a particular roadway can be increased dramatically. In the design of crest vertical curves there is, as to be expected, a tradeoff between available sight distance and the cost of excavation or fill. Longer stopping sight distances lead to longer vertical curves that, in turn, increase construction costs.

The current procedures for determining required stopping sight distances are intended to allow for a normally alert passenger-car driver, traveling at or near the highway design speed on a wet pavement, to react and bring their vehicle to a stop before striking a stationary object in the road. The values in the American Association of State Highway and Transportation Officials (AASHTO) model for desirable stopping sight distances are based on an initial speed equal to the design speed of the roadway, while the values for minimum stopping sight distances are based on the premise that most motorists drive slower than the design speed (i.e., an initial speed less than the design speed). The model used in determining required stopping sight distance was formalized in 1940 by the then American Association of State Highway Officials (AASHO) (*1*). Over the past 50 years, the model's parameter values have been altered to compensate for changes in eye height, object height, and driver behavior; however, recent research has raised questions as to the appropriateness of several parameters within the current AASHTO stopping sight distance model (*2, 3, 4*).

The minimum length of vertical curves is controlled by the following three parameters - required stopping sight distance, driver eye height, and object height. The resultant curve length provides sight distances longer than or equal to the required stopping sight distance along the entire length of the vertical curve. The object height, one of the three parameters, was established as 100 mm (4 in) in 1940, and was changed to 150 mm (6 in) in 1965. In both cases, the object height value selected was based on judgment rather than research results. Because this parameter directly affects vertical curve lengths, there is a need to determine an appropriate object height for use in the geometric design of highways, as well as a model that appropriately reflects human visual capabilities.

The objective of this study was to evaluate driver visual capabilities in detecting and recognizing objects of various sizes and contrasts. The evaluation was conducted to determine if drivers could detect and recognize an object under different lighting conditions at or beyond the minimum stopping sight distance required by AASHTO. To accomplish the objectives of this study, the following tasks were performed:

- Review relevant literature concerning the object height parameter in the stopping sight distance model and relevant studies on object visibility and driver visual capabilities;
- Develop a study plan to measure driver capability in detecting and recognizing objects of various sizes and contrasts under different lighting conditions in a controlled testing environment;
- Collect the field data for the necessary analysis;
- Analyze the data to determine the visual capabilities of drivers, including detection and recognition distances; the effects of lighting conditions on visual capabilities; and the effects of low- and high-beam vehicle headlights on drivers' visual capabilities at night.

This appendix on *Study 2 – Determination of Driver Visual Capability in Object Detection and Recognition* is divided into five sections. The first section includes the introduction, problem statement, and the research objective. Section two presents a background review of the object height in the SSD model, as well as a review of relevant studies that have been conducted, including object visibility and driver visual capabilities. The third section includes the study design which describes the procedure for collecting the driver visual capability data. The results of the driver visual capability studies are presented in section four. The last section presents the conclusions and recommendations from this study.

LITERATURE REVIEW

One of the most important requirements in highway design is the provision of adequate stopping sight distance at each and every point along the roadway. Horizontal and vertical curves can limit a driver's available sight distance; however, when they are designed in accordance with AASHTO criteria, available stopping sight distance at every point along the curve is greater than or equal to the minimum stopping sight distance. Designing vertical curves using

AASHTO's minimum stopping sight distance allows a "below average" driver to detect an unexpected, stationary object in the road and to stop the vehicle before striking the object (*3*).

This section reviews the AASHTO "object-in-the-road" model, the model's changes over time, and object visibility and driver visual capability studies relevant to the objectives of this study. Headlight visibility characteristics and limitations relevant to the stopping sight distance situation also are reviewed. Each of these topics is discussed in the following sections.

AASHTO'S STOPPING SIGHT DISTANCE MODEL

The visual requirement for a driver is to not only see the roadway ahead, but also to see an object in the roadway ahead. This requirement is referred to as sight distance, or the length of roadway ahead visible to the driver. There are currently three types of sight distance used in roadway design—stopping sight distance, passing sight distance, and decision sight distance. Stopping sight distance (SSD) is the most critical of the three sight distances because it represents the minimum design values. It is defined as the "minimum sight distance required for a driver to stop a vehicle after seeing an object in the vehicle's path without hitting that object" (*2, 3, 4*). For roadway design purposes, the determination of required stopping sight distances under current AASHTO procedures incorporates a number of driver, vehicle, and roadway parameters.

In 1940 the American Association of State Highway Officials (AASHO) developed seven national policies that recognized certain aspects of geometric design (*1*). Prior to this time, several state policies were in existence but no nationwide standards had been established; furthermore, it is believed that the issues of geometric design were not thoroughly understood (*5*). The stopping sight distance model developed in 1940 represented significant changes from previous highway design practice in that a mathematical model with several driver, vehicle, and roadway parameters was established to describe the stopping sight distance situation. The design parameters in the SSD model, still in use today for design purposes, include the perception-response time (PRT) of the driver, the velocity of the vehicle, and the tire-to-pavement friction ratio. Additional parameters for vertical curve design include the driver eye-height, the stationary object height, and the algebraic difference in grades of the approach and departure roadways (*3, 6*).

The fundamental stopping sight distance equation is based on the basic principles of physics and the relationships between the various design parameters. AASHTO defines stopping sight distance as the sum of two components--the brake reaction distance (distance traveled from the instant of object detection to the instant the brakes are applied) and the braking distance (distance traveled from the instant the brakes are applied to when the vehicle is decelerated to a stop) (*3*).

Stopping sight distance can be expressed by the following equation:

SSD = Brake Reaction Distance + Braking Distance [1]

More specifically, these two components can be mathematically expressed as follows:

$$SSD = 0.278\,V t_{pr} + \frac{V^2}{254.3(f+G)} \qquad \left[1.47\,V t_{pr} + \frac{V^2}{30(f+G)}\right] \qquad [2]$$

where:

- SSD = required stopping sight distance, meters [feet];
- V = design or initial speed, kilometers per hour [miles per hour];
- t_{pr} = driver perception-response time (PRT), seconds;
- f = friction coefficient between the tires and the pavement surface; and,
- G = grade, expressed as a decimal.

Minimum stopping sight distance values are used to calculate the required length of horizontal and vertical curves. As noted, the minimum length of vertical curves is controlled by the required stopping sight distance, the driver eye height, h_e, and the stationary object height, h_o. This required length is such that at a minimum, the stopping sight distance calculated by Equation 2 is available at all points along the curve. Figure D-1 illustrates the SSD model parameters as they relate to crest curve geometry.

HISTORICAL DEVELOPMENT

Although the basic stopping sight distance model has remained the same over the past 50 years, there have been a number of changes and much discussion regarding parameter values within the model because of changing human, vehicle, highway, and environmental considerations (*5*). For example, the object height, one of the least sensitive parameters in the SSD model (*5*) and the subject of this paper, has been a much debated topic over the past ten years. The following sections of this working paper discuss the history of the object in the SSD model and the significance of the changes in object height over the last half century.

History of the Object

The need for adequate stopping sight distance on roadways was recognized as early as 1914 (*7*); however, it was not until 1921 that Harger (*8*) incorporated lengths and heights to the sight distance concept. He argued that a clear line of sight of 350 feet was reasonable for Main Commercial Special Service roads and a clear line of sight of 250 feet was

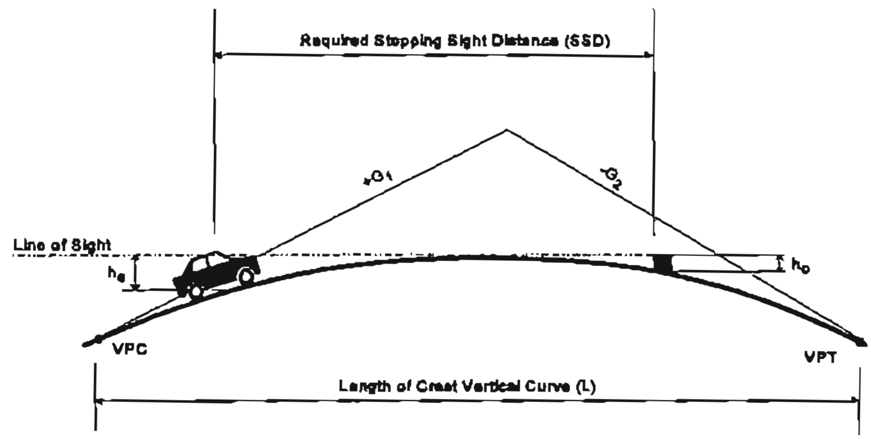

FIGURE D-1. AASHTO's Stopping Sight Distance Model on a Crest Vertical Curve.

reasonable for Local Service roads. Line of sight was defined as a tangent connecting two points 5.5 feet above the ground on both ends. Harger also presented minimum lengths of vertical curves that were based on providing this clear line of sight for a range of algebraic difference in grades.

During the 1920s and 1930s, the Bureau of Public Roads and several states recognized the importance of sight distance in their various publications, but each agency used a slightly different standard; i.e., there was no national consensus. In 1936, Gutmann (*9*) presented the German standard for stopping sight distance that was being used in the construction of their superhighways during that time period. His equation, similar to that used by AASHTO today, is shown on the following page:

$$L = 1.47V + \frac{V^2}{f \pm \tan\alpha} \qquad [3]$$

where:

L = sight distance, feet;
V = speed, mph;
f = coefficient of friction, assumed to be 0.4 or 0.5; and,
α = grade angle.

Perception-response time does not appear in the 1936 German equation because it was assumed to be equal to 1.0 second. When computing required lengths and minimum stopping sight distance for crest vertical curves, the height of the driver's eye was assumed to be 1.2 m (3.9 ft) above the ground and two kinds of obstacles were considered -- a standard car, 1.5 m high (4.9 ft), and an object near the summit projecting 200 mm (8 in) above the surface of the roadway. These requirements were for level terrain. For irregular terrain and speeds of 140 km/h (87 mph), a maximum sight distance requirement of 200 m was somewhat arbitrarily chosen; however, this distance could have been shortened if excessive costs were a problem.

During this same time period, engineers in the United States were debating the type of encounter and/or object that a driver was expecting in a stopping sight distance situation. Many agencies using sight distance criteria considered an approaching vehicle to be the critical encounter; other agencies argued for smaller objects. In 1940, AASHO's stopping sight distance publication (*1*) formalized the critical obstacle to be a four-inch object, the so-called "dead cat" rule (*10*). The change was a slight modification of the German criteria for stopping sight distance that was previously presented; i.e., the object height was reduced from eight inches to four inches. Interestingly, the 1940 AASHO policy did not explicitly present an equation for stopping sight distance. Rather, it explained the concept of brake reaction distance and braking distance, and the factors that influenced these distances.

Required stopping sight distance was defined in the 1940 Policy as the distance required to stop a vehicle before reaching a stationary object in the same lane when the vehicle was traveling at the assumed design speed. "The stationary object could have been a vehicle or a high object, but it could also have been a very low object such as merchandise dropped from a truck or small rocks from side cuts (*1*)." The 1940 Policy noted that the surface of the roadway would have provided the safest design, but an object height of four inches was chosen because large holes in modern pavements were uncommon and other very small objects generally can be avoided without the necessity for stopping. Thus, the object height chosen in 1940 was based on logic and engineering judgment.

Changes Over Time

The 1954 AASHO Policy, *A Policy on Geometric Design of Rural Highways* (*11*) changed the definition of stopping sight distance to include the distance required to stop from the instant that the object became visible. The 1954 Policy provided further support for the choice of the four-inch object height by stating that a smaller object (zero object height) was not justified because of the high construction costs, and a larger object would exclude lower hazards and produce dangerously short lengths of vertical curves. In addition, AASHO noted a significant relationship between object height and vertical curve length -- the length of the vertical curve decreased rapidly as the object height was increased from zero to four inches; for greater increases in object height, the decrease in length of vertical curve was less significant.

For example, the required length of vertical curve decreased by 38 percent when going from a zero to four-inch object height, by 48 percent when going from a zero to eight-inch object height, and by 60 percent when going from a zero to eighteen-inch object height. These percentages were for the situation where the curve length exceeds the required stopping sight distance. Thus, changing the object height from zero to something higher than four inches resulted in shorter curve lengths, but at the expense of decreasing the driver's ability to see conditions on the roadway ahead. AASHO concluded that the four-inch object offered a compromise between the cost of excavation and the ability of the driver to see the road ahead. "A four-inch control was considered the approximate point of diminishing returns (*11*)."

In the 1965 AASHO Policy, *A Policy on Geometric Design of Rural Highways* (*12*), the driver eye height was decreased from 4.5 feet to 3.75 feet and the object height was increased to six inches. The same rationale and wording (i.e., "approximate point of diminishing returns") were used to justify the six-inch object in 1965 as were used to justify the four-inch object in 1954. With the new values for driver eye and object height, the decrease in required curve length for a change in object height from zero to six inches was 46 percent. For a change in object height from zero to eighteen inches, the decrease in the required curve length would only have been 63 percent. Because of the relatively slight decrease in the required vertical curve length (46 to 63 percent) for such a large increase in object height (six to eighteen inches), the six-inch object was referred to as the point of diminishing returns.

In the 1984 AASHTO Policy, *A Policy on Geometric Design of Highways and Streets* (*2*), the driver eye height was reduced to 3.5 feet because of continuing decrease in vehicle heights. Object height remained the same. The combination of this change and others between the 1965 and 1984 AASHTO Policies, increased required vertical curve lengths by small amounts at low design speeds and larger amounts at high design speeds; i.e., 25 percent for 40 miles per hour and 111 percent for 70 miles per hour. Somewhat surprisingly, the explanation for using the six-inch object changed slightly from 1965 to 1984 although the height remained the same, i.e., from the point of diminishing returns in 1965, to the smallest object that could create a hazardous situation in 1984.

The 1984 and 1990 Green Books (*2, 3*) emphasized that a six-inch object represented the lowest object that could create a hazardous situation. They go on to state that an object height of six inches was "largely an arbitrary rationalization of possible hazardous objects and a driver's ability to perceive and react to a hazardous situation" (*2, 3*). Using an object height lower than six inches would produce vertical curves lengths up to 85 percent longer and significantly increase construction costs due to additional excavation. Using an object height higher than six inches would produce slightly shorter vertical lengths, but decrease the driver's ability to see conditions on the roadway ahead.

National Cooperative Highway Research Program Report 270 (*13*) was published the same year as the 1984 AASHTO Green Book. It recommended that the object height be decreased from six inches to four inches, thus increasing the required length of the vertical curve by an additional 10 percent. The recommendation was made based on a paper by Woods (*14*) which stated that 30 percent of vehicles had under clearances that were less than six inches and that all vehicles had under clearances that were greater than four inches. The recommended increase in vertical curve length was not quantitatively supported by safety or cost data. Rather the recommendation was based on the ground clearance of a vehicle (*13*). Because there was no data supporting the need for such a change, it was not adopted by the highway design community.

A summary of the different object height values in the early engineering textbooks and the seven AASHTO Policies is shown in Table D-1. Before 1940, the object height was generally assumed to be the same as the driver eye height, which is considerably greater than the four-inch and six-inch object heights adopted in the 1940 and 1965 AASHO Policies, respectively. In 1940, the object height selected offered a compromise between the cost of excavation and the ability of the driver to see the road ahead; however, in 1984, the rationale for the object height selected changed to the smallest object that could create a hazardous situation. The 1994 AASHTO Policy established the object height at 150 mm (5.9) in response to metrication requirements (*4*).

TABLE D-1. History of the Object Height Parameter.

Source	Date	Object Height Millimeters	Object Height Inches
Harger (*8*)	1921	1676	66.0
Agg (*15*)	1924	1524	60.0
Michigan (*16*)	1926	1524	60.0
Oregon (*17*)	1935	1524	60.0
Wiley (*18*)	1935	1524	60.0
Conner (*19*)	1937	1524	60.0
Bateman (*20*)	1939	1524	60.0
Agg (*21*)	1940	1524	60.0
AASHO Policy (*1*)	1940	102	4.0
AASHO Policy (*11*)	1954	102	4.0
AASHO Policy (*12*)	1965	152	6.0
AASHTO Policy (*22*)	1970	152	6.0
AASHTO Policy (*2*)	1984	152	6.0
AASHTO Policy (*3*)	1990	152	6.0
AASHTO Policy (*4*)	1994	150	5.9

Significance of the Change in Object Height

The change in the object height from four inches to six inches in the 1965 AASHO Policy (*12*) was not based on an analysis of the object as a potential hazard. Additionally, there was nothing in the safety literature that supported the change. So why was the object height changed? If the object height had remained at four inches when the driver eye height was decreased to 3.75 ft, the length of the crest vertical curve would have increased by 15.3 percent; however, with the changes in both object and driver eye heights, the required vertical curve lengths increased by only 4.2 percent. Thus, whatever the intent, the end result was relatively constant vertical curve length requirements.

As mentioned, the same explanation that was used to justify the four-inch object in 1954 was used to justify the six-inch object in 1965: the point of diminishing returns between excavation costs and visibility. Figure D-2 shows the percent reduction in vertical curve length for different object heights in the 1954 and 1965 SSD models. Note that going from zero to a four-inch object in 1954 resulted in a 38 percent reduction in vertical curve length and that going from zero to a six-inch object would have resulted in a 44 percent reduction in vertical curve length (a six percent difference in vertical curve length). Note also that the two curves are very similar in shape and do not suggest that the point of diminishing returns had changed - going from a zero to a six-inch object in 1965 resulted in a 46 percent reduction in vertical curve length and going from a zero to a four-inch object would have resulted in a 41 percent reduction in vertical curve length (a five percent difference in vertical curve length.

In the late 1960s, Glennon (*23, 24*) suggested that the point of diminishing returns appeared to have a more appropriate balance in the range of 30 to 90 mm (1.2 to 3.6 in). Glennon went on to state that excavation was no longer the major cost of highway construction and operational safety was a greater concern when designing vertical curves; however, there was no safety data to support his suggestion of a lower object height. In a following paper, Loutzenheiser (*23*) disagreed with Glennon's views on a lower object height. Loutzenheiser defined diminishing returns to be the point where a relatively large expenditure was needed to purchase a relatively small benefit. In terms of stopping sight distance, the length of the vertical curve represented expenditure and the safety from the lower object height represented the benefit. He suggested that the object height could have been higher than Glennon predicted because the earthwork expenses increased exponentially as the lengths increased.

It is important to note that the two curves in Figure D-2 illustrate the relationship between curve length and visibility of the roadway ahead -- the shorter the curve, the less the visibility. They do not illustrate the relationship between curve length and safety. Although the existence of such a relationship is intuitive, it has never been quantified. In other words, the point at which limited visibility becomes a safety problem was never documented prior to the 1990s. Working Paper 4, *Accident Causation Study on Roadways with Limited Stopping Sight Distance*, suggests that on high speed roadways, accidents began to increase between 100 and 120 m of stopping sight distance. Thus, as long as the selected object height provides these sight distances as a minimum, increased accident rates should not be an issue.

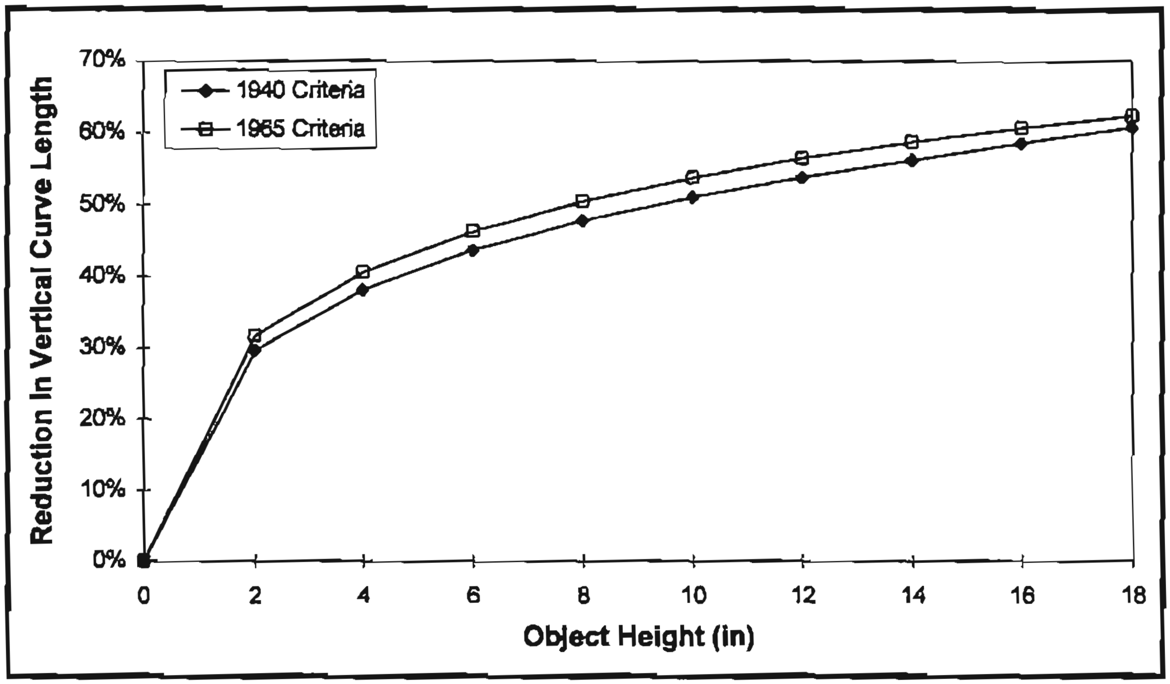

FIGURE D-2. Sensitivity of Vertical Curve Length to Object Height (S>L).

OBJECT VISIBILITY

Surprisingly, the driver's visual capabilities were not considered when the object was chosen for the stopping sight distance model. Hall and Turner (*5*) expressed concern for the driver's ability to see a six-inch object at the required stopping sight distance on rural high-speed highways. They determined that at a distance of 600 ft, a driver with 20/40 static visual acuity is required to see an object that is much smaller than their vision allows; i.e., at 600 ft a six-inch object would have to be 3.5 times larger (21 inches) to be visible to the driver. Thus, perception and/or recognition of small objects may be beyond the driver's visual capability at long sight distances.

The following sections of this paper present a review of the literature on the human visual process and visibility and target acquisition; identifies concerns about the drivers' line of sight to the object and their capability to detect and recognize a small object; and summarizes studies of driver visual capabilities and reactions to small objects in the roadway.

Human Visual Process

The human visual process is complex in that it requires visual information, visual scanning and fixation by the driver, and a cognitive involvement of the brain. The visual scanning and fixation are a continual process that allows a person to detect and recognize visual information in the foveal (line of sight) and peripheral fields of vision. Contrast determines the limit for object detection and size determines the limit for object recognition.

In order to accomplish visual detection and recognition, the retina in the eye acts as a photographic plate. Sharp vision is confined to a very small area in the center of the retina that subtends one to two degrees of arc either side of the forward field of view. The retina defines visual information primarily by means of 5.5 million cells called cones. Cones are color-sensitive, respond to bright light and color detail. The visual process is functioning at the highest level of sensitivity during the day (bright light conditions). Visual acuity, color discernment, depth-perception, glare recovery, and peripheral vision are the most sensitive during this time. Bright light conditions afford the greatest capability for distinguishing differences between objects in the visual field making this part of the visual process, photopic vision, important for the driving task.

At night, the eyes rely primarily on another set of 125 million cells called rods, which are sensitive to small amounts of light. Rods are not color-sensitive; i.e., "rod vision" is in shades of gray to black. This part of the visual process is known as scotopic vision and is of negligible importance for driving. A middle range of light level in which both rods and cones function is known as mesopic vision. Typically, night driving is done under mesopic conditions depending upon the amount of light available (*25*). It is important to note that because of headlights and other light sources at night, drivers rely on their photopic vision to see objects in the visual fields; i.e., rods are only useful in completely dark conditions.

Visual Requirements for Driving

There are certain minimum visual requirements for all motor vehicle operators. For example, to acquire and renew a driver's license, drivers must pass a minimum static visual acuity test, which tests their ability to detect stationary objects and legend messages. Minimum visual acuity requirements for driving are established by the states. If a person fails to meet the minimum requirements, they are required to wear corrective lenses while driving.

The minimum visual requirement established by most state licensing agencies is based upon a driver's ability to discern, in a laboratory-type situation, static visual targets and/or text messages. For driver licensing purposes, a Snellen Visual Acuity rating of 20/40 is a typical visual acuity level before corrective lenses are required. The Snellen rating is based on a person's ability to discern an object at a particular visual angle (VA). The visual angle is the angle subtended by the size (height) of an object or some other object dimension (i.e., stroke width of a letter) at some distance from the eye in the foveal field of vision. The relationship between the visual angle, object height, and distance is shown in Figure D-3. The visual angle (α) used to determine a person's visual acuity can be calculated as follows:

$$\alpha = 2 \times \arctan \frac{L}{D}$$

where:

α = visual angle, minutes of arc;

L = diameter of the target (letter or target); and

D = Distance from the eye to the target in the same units.

All things being equal, two objects that subtend the same visual angle will elicit the same response from a human observer, regardless of their actual sizes and distances. The Snellen eye chart visual acuity ratings are related to the size of objects in terms of visual arc, radians (equivalent for small sizes to the tangent of the visual arc) and legibility indices in Table D-2.

Jacobs, Johnston, and Cole (*26*) found that 27 to 30 percent of the driving population cannot meet a 20/20 criterion and that most states have a 20/40 static acuity criterion for unrestricted licensure, and accept 20/60 for restricted (daytime usually) licensure. Also, such tests in driver license offices are subject to error, and examiners tend to be very lenient. Nighttime static visual acuity tends to be at least one Snellen line worse than daytime, and much worse for older drivers.

More relevant to the driving task than static visual acuity is dynamic visual acuity, depth perception, glare recovery, and peripheral vision. These visual characteristics are not generally tested during licensing exams, but are important for visually distinguishing between objects in the visual field. Because of the inherent difficulties of quantifying their values, however, their relationship to the driving task is not well understood. Drivers may or may not compensate if these characteristics have deteriorated; e.g., by not participating in activities that may hinder their visual capabilities (i.e., alcohol and drug use), by driving slower and more cautiously at night, and by being more attentive while driving.

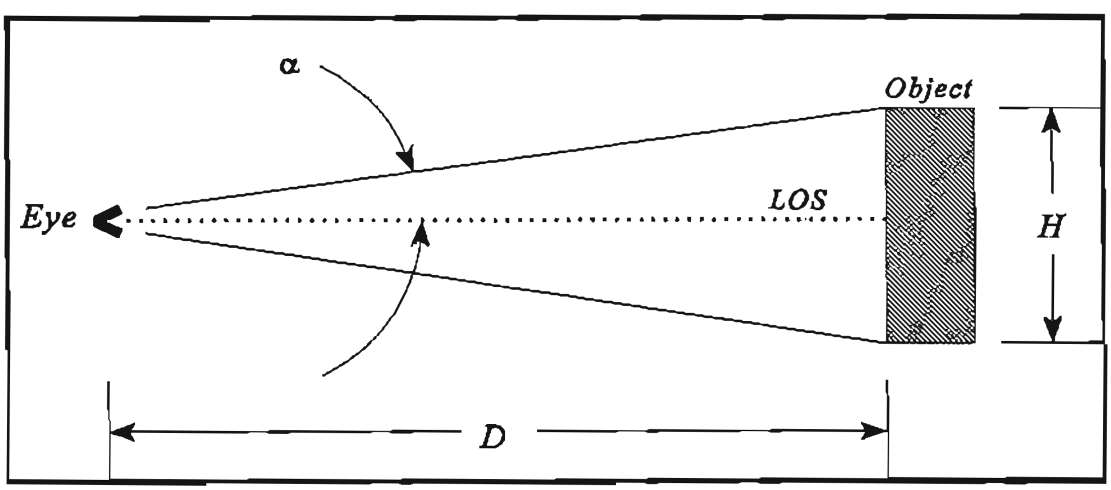

FIGURE D-3. Relationship Between Visual Angle and Object Height.

TABLE D-2. Visual Acuity and Letter Sizes.

Snellen Acuity	Visual Angle of Letter or Symbol		Legibility Index	
SI (English)	Arc (min)	Radians	SI (m/cm)	English (ft/in)
6/3 (20/10)	2.5	0.00073	13.7	114.6
6/6 (20/20)	5.0	0.00145	6.9	57.3
6/9 (20/40)	7.5	0.00218	4.6	38.2
6/12 (20/50)	10.0	0.00291	3.4	28.7
6/15 (20/50)	12.5	0.00364	2.7	22.9
6/18 (20/60)	15.0	0.00436	2.3	19.1

Visibility of Targets

Besides the driver visual process, there are also a number of psychological and physical factors involved in object detection and recognition. Psychological factors are task-dependent and are strongly influenced by the environment. These factors include object novelty, relevance to the driving task, expectancies, workload level, and familiarity with the driving environment. Physical factors that can have either a positive or negative effect on holding a driver's attention in the field of vision include effective object size, luminance and color contrast, texture (contour and complexity), shape, dynamics, location, and time exposed (*25*).

With regard to object visibility, the object in the stopping sight distance model can be referred to as a target. Target acquisition is dependent upon the psychological factors presented earlier, but more important, the physical factors of the object. Boff and Lincoln (*27*) identify these factors as the dimensions of the target (diameter, area, shape), brightness (luminance) and color contrast, texture gradients (boundaries), dynamics, environmental factors of illumination (glare), sun angle, and atmospheric conditions such as humidity or haze.

Providing sufficient light from the vehicle's headlamp system for detection and recognition of objects at night is complicated by the fact that headlamps illuminate both the target object and the target's background. When objects are viewed against backgrounds with similar reflectance characteristics, the contrast between the target and the background may be insufficient for detection and/or recognition until the vehicle is too close to the object for the driver to stop or make an appropriate evasive maneuver (*28*). Thus, in order for objects to be adequately seen at night, the objects must be sufficiently brighter or darker than their background (*25*).

Brightness or luminance contrast refers to the difference in brightness between the target and the background against which it is viewed. The brightness difference is the crucial factor for target acquisition. There must be a difference in brightness between the target and the background for the object to be detected, unless color differences exist. The equation generally used for luminance contrast is:

$$L = \frac{l_b - l_t}{l_b} x\ 100 \qquad [5]$$

where:

L = the luminance contrast, percent;
l_b = the luminance of the background, (nits); and
l_t = the luminance of the target object, (nits).

The critical size of the detectable object decreases with increasing illumination. The luminance difference must be large enough to allow form perception or object conspicuity. This critical value is called the suprathreshold factor.

Texture refers to the variations in brightness per unit of visual angle. Texture contrast refers to the difference between the target and its background with respect to the textures of the two surfaces. If the target has a negative contrast, then the object is lighter than the background. Positive contrast occurs when the object is darker than the background (*29*). Texture gradient refers to the transition between the target texture and the background texture. This gradient may be abrupt and sharp, or it may be gradual and fuzzy.

The shape of a target in the context of this discussion is related to whether the shape is immediately recognizable to an observer. For faint or brief targets, uncertainty about the spatial location or the size of the target leads to difficulty in detection. If the object is rectangular, then visibility increases when the longer dimension increases, up to approximately 40 to 50 minutes of arc. Visibility is also increased when the shorter dimension of the rectangle is lengthened up to approximately 5 minutes of arc (*27*). The oblique effect states that elongated targets have higher visibility thresholds when they are at oblique angles to the direction of travel than when they are oriented perpendicular to the direction of travel. The visibility threshold is the point where the object can be seen, but not yet recognized.

Stray light produced by sources of high illuminance creates glare which impairs the drivers vision (*29*). If glare is a problem, then the luminance difference between the object and background must be larger than usual to keep the same visibility.

Line of Sight to the Object

Hall and Turner (*5*) questioned the assumption that the driver only needs to see the top of the object to recognize it, react, and stop. They calculated that in the distance and time that it takes for the entire six-inch object to become visible at 60 miles per hour, the available stopping sight distance has decreased by 165 feet. Hall and Turner even questioned whether height is enough to describe the critical obstacle that should be visible to the driver.

American and British design guides imply that the moment the top of the object comes into view it is visible to the driver; however as mentioned, other visibility factors must be considered before the object can be assumed to be visible: luminance contrast, color contrast, ambient luminance level, and glare. Hills offered two conclusions concerning the visibility of the object (*30*). First, it was the portion of the object above the specified object height that the driver responded to, called the "object cut off height," and second, objects of the same height were not equally visible. For example, a vehicle rooftop would be more visible than a child. For these reasons, Hills stated that the line of sight should not be equated with visibility.

The Design Standards in Sweden (*31*) address the problem of object recognition by specifying an obstruction height and a visibility angle. The obstruction height of 200 millimeters (eight inches) is the perpendicular distance from the top of an obstruction to the roadway surface. "The obstruction must be visible to a normal eye (*31*)." Under bright light conditions, one minute of arc is the minimum angle that part of the obstruction must cover to allow a driver with 20/20 static visual acuity to perceive it as an object.

The visible portion of the obstruction at one minute of arc is subtracted from the obstruction height to obtain the effective object height. The portion of the obstruction that must be visible on a crest vertical curve is dependent on the distance to the object and the speed of the vehicle. Table D-3 shows the speed, sight distance, visible portion, and effective height of the object. The effective height of the object is close to AASHTO's six-inch object; however, Sweden has first presumed that the driver must detect a portion of the object before they can recognize and react to it.

McLean. McLean (*32*) identified the drivers' visual limitations within the stopping sight distance model in an evaluation of the 1980 National Association of Australian State Road Authorities (NAASRA) Geometric Design Guide. McLean stated that an observer could resolve detail under ideal lighting and contrast conditions when it spanned one minute of arc. Considering the conditions that a driver was exposed to on the roadway, five minutes of arc would be necessary to perceive an object on the roadway surface.

Using these conclusions, 100 millimeters of an object must be above the line of sight to detect it at a distance of 65 meters. At a distance of 130 meters, the object must have 200 millimeters above the line of sight to be detected. "For distances greater than 130 meters, it was likely that the design object (NAASRA object height equaled 200 millimeters, or approximately eight inches) would not be seen even with completely clear sight distance (*32*)." McLean hypothesized that sight distances required for speeds above about 90 kilometers per hour were beyond the visual capability of the driver to detect the hazard.

TABLE D-3. "Visible" Height and Effective Height of Object - Sweden.

Height	Speed, km/h (mph)			
	50 (31)	70 (44)	90 (56)	110 (68)
Sight Distance, m (ft)	70 (230)	120 (394)	165 (541)	195 (640)
Visible Portion, mm (in)	20 (0.79)	35 (1.37)	50 (1.89)	55 (2.23)
Effective Height, mm (in)	180 (7.07)	165 (6.50)	150 (5.98)	145 (5.64)

Object Visibility Tests

Ketvirtis studied the major factors that contributed to the visibility needs in detecting a critical size object at a safe stopping sight distance (*33*). Four separate topics were considered in his study: the visual capability of the drivers, the characteristics of the object, the quality of light acting as an intermediary, the qualities of the pavement, and then the interaction between these four areas.

Ketvertis first attempted to define the critical object that was visible at the safe stopping sight distance because it represented the most critical visibility condition. The psychological impact and physical reality of the objects were also studied. Because the average passenger vehicle had an undercarriage clearance of approximately 180 millimeters (seven inches, *34*), the critical dimension of a hazard was chosen as the height. The volume of the object was considered significant, but the weight of the object and the type of material could not be determined at the required stopping sight distance. The width was not considered critical because the average vehicle traveled within a wheel path of 4.5 feet. Based on the width of the lane and the wheel path, objects located within 1.9 feet of either edge of the lane or within 2.0 feet of the center of the lane could be avoided if they were narrower than 1.6 feet.

The drivers' reactions and the object detection distances were tested during both the day and night with a study that used 12 objects. The object sizes ranged from 60 by 90 by 270 millimeters up to 610 by 200 by 150 millimeters. Drivers' responses were also tested with a video made along a roadway where objects were encountered. They were asked at what distance they would make a decision to react to an object in the roadway and how they would rate the object as a potential hazard. When given the option to stop, go around, or pass over, very few observers chose the full stop option. Many observers chose to go around a 150-mm object or higher and almost all chose to pass over a 100-mm high object or lower. The observers, however, were instructed that the roadway volume was very low which allowed the option to safely go around the object (*33*).

When the objects were rated from the film, 100-mm objects were rated as a minor hazard and 200-mm objects were rated as a moderate hazard. The scale ranged from no hazard, minor hazard, moderate hazard, hazard, to high hazard. A muffler with a height of 150-mm was rated as a hazard probably because of its length of 610 millimeters. The two animals used as objects had lower hazard ratings than other objects of the same size height.

Ketvirtis presented three conclusions from these two studies:

- The degree of hazard is often overrated, especially at night;
- An object of 100 to 120 millimeters (4.0 to 4.7 inches) may precipitate a response of moderate degree (e.g. initiate the use of brakes or a change of lane); and
- The actual smallest size to constitute a physical hazard is approximately 200 millimeters (approximately eight inches).

The second part of Ketvertis' study was to establish the visibility distance of objects under different luminance levels. The distances were based upon detection only and the following three objects from the previous study were used: one 300 by 200 millimeters square, and two 610 by 200 by 150 millimeters blocks. The results showed that a luminance level of 1.0 cd/m^2 (0.09 candela/ft^2) was needed to see an object at approximately 160 meters (525 feet). Luminance above 1.2 cd/m^2 (0.11 cd/ft) added little to the visibility distance.

The final part of the study was performed at an outdoor lighting laboratory where the experiment investigated luminance, uniformity, glare, target reflection, and target size during night conditions. The study showed that luminance was one of the most important light factors. Uniformity of the background, object, and pavement had a direct influence on object detection. Object size and glare also had an effect on the visibility. The field test results showed that under night driving conditions, the background luminance should be approximately 1.0 cd/m^2 (0.09 cd/ft) to see a critical object at a distance of 160 meters (525 feet) and a speed of 100 kilometers per hour. Driver reactions, however, suggested that at 130 meters the driver could react and make required adjustments in their driving task to avoid a hazard, with a minimum of 0.8 cd/m^2 (0.07 cd/ft^2) of luminance.

HEADLIGHT VISIBILITY LIMITS

The basic performance requirements that all headlamps are designed to meet are contained in Federal Motor Vehicle Safety Standard 108 (*34*) and Society of Automotive Engineers (SAE) J579 (*35*). These requirements apply to all vehicles registered in the United States, regardless of the design of the headlamp filament or light source. Two-headlamp systems use Type 2 and Type 2A (rectangular) headlamp units, both of which must meet the same requirements. Four-headlamp system uses two Type 1 or 1A, and two Type 2 or 2A headlamp units. Two and four-headlamp systems have the following performance ranges:

Type 2 or 2A Sealed Beam
  Upper Beam (Each lamp): 20,000 to 75,000 candela
  Lower Beam (Each lamp): 15,000 to 20,000 candela.

Type 1 or 1A Sealed Beam
  Upper Beam (Each lamp): 18,000 to 60,000 candela.

These illumination levels are the "hot spot" levels for each type of lamp. The illumination levels fall off rapidly from these maximum ranges as the pattern diverges in any meridian or eccentricity from the nominal hot spot. Because these headlamp arrays are not aimed to coincide or merge at a distance, but rather provide an extended illuminated field with either two or four hot spots, a given object will be illuminated to a first order of approximation by a single headlamp. Research by Bhise (*36*) at Ford Motor Company suggests that variations by as much as a factor of two are routinely encountered in highway situations. Low voltages and the use of many accessories drive the illumination levels downward, whereas high charging rates and over-voltages drive the illumination levels upward (to the detriment of lamp life).

Nominally, high beams are aimed to provide the hot spot beam in a direction parallel to a level highway surface with the vehicle unloaded. Low beams are aimed to place the hot spot several hundred feet ahead of the vehicle and at a location somewhat to the right of directly ahead, although the pattern is more complex than this description. The principal hot spot for low beam headlamps is specifically aimed 0.5 degrees below the straight-ahead (horizontal/vertical reference point) and 1.5 degrees to the right. At a nominal 600 millimeters (1.9 feet) above the pavement (the minimum headlamp height), the low beam hits the pavement at a distance of 70 meters (230 feet), with a deviation or eccentricity of 1.8 meters (6 feet) to the right. Obviously small variations in mounting height can make considerable differences in the distance at which the hot spot hits the pavement. A 125-millimeter (5-inch) difference in height results in 15 meter (50 feet) or more change in range.

Illumination Levels and Other Considerations

At a nominal distance of 70 meters (230 feet), a maximum (legal) level high beam lamp would provide an illumination of 1.4 lumen/ft^2. A low beam lamp would deliver a maximum of 0.4 lumen/ft^2, at the pavement surface. Other aspects of headlamp visibility which should be considered in stopping sight distance include the prevalence of tinted windshields. Many vehicles come equipped with these windshields to ease the cooling load on hot days. Typical transmission values for such windshields run in the neighborhood of 65 percent (*39*).

The reflectivity of pavements varies considerably, and has not been measured very accurately. Generally, objects on a highway are characterized by low contrast. Percentages of retro-reflectance at distances of 200 to 400 feet from the driver tend to be in the 10 to 20 percent range (*36*). Reflectance of common highway backgrounds is shown in Table D-4.

Empirical Studies of Headlamp Visibility

Empirical studies performed between 1938 and 1982 with low beam headlamps that would probably meet today's standards yield some ranges of visibility distance for two general types of objects (large and small) and two driving conditions, with and without opposing headlamp glare (*37*). Twenty-five such studies were described, with 3 to 19 studies in each of the cells of this two-by-two classification scheme. Table D-5 summarizes the 25 studies. Each cell entry under *Distances* gives the range of mean sight distances to object detection (visibility distance) for that combination of object and glare condition. Variations in procedures, equipment, and objects account for the ranges of values reported, but an important consideration is how these values compare to AASHTO's stopping sight distance values. A safety factory can be described as the ratio of visibility distance to stopping sight distance. Table D-5 lists the safety factors calculated using a stopping sight distance of 450 feet (minimum stopping sight distance for a 55 mph design speed) and the empirical headlight visibility distances reported by Perel, et al. (*37*). Safety factory less than one denote a hazard because the visibility distance is less than the braking distance. Only one study involving a small object under no glare meets this criterion.

A more recent summary by Olson and Sivak (*38*) points out that expectation plays a major role in visibility distances under nighttime driving conditions. Distances are reduced by half when subjects had no idea that they were to detect objects, such as pedestrians or roadway obstacles, in front of them as they drove. They conclude that a reasonable range for car drivers is 0 to 180 feet, all things considered. They go on to say that a rough rule of thumb for the improvement that might be expected with the use of high beams is of the order of 50 percent on the right side, and 100 percent (doubles the visibility distance) on the left side. It will be recalled that low beams aim slightly to the right, whereas high beams are aimed straight ahead. They provide a useful summary table which is reproduced as Table D-6. In this Table, the objects are all large, pedestrians, dressed in light or dark clothes. "Response distance" is defined as the distance at which the drive might initiate a control input, such as "hazards." In the context of this report, response distance is stopping sight distance. Note that none of these distances come even close to 450 feet AASHTO estimate for stopping sight distance at 55 mph.

TABLE D-4. Reflectance of Common Highway Backgrounds (2).

Common backgrounds	Reflectance*
Pine trees	.02 - .08
Grass: Long, dormant, pale green	.08 - .16
Grass: Lush green, closely mowed	.10 - .18
Forest: Mixed green	.02 - .26
Dirt: Packed, yellowish	.23 - .43
Asphalt: Oily with dust film	.06 - .13
Concrete: White, aged	.25 - .37
Pedestrians: Median of 54 winter coats	.055
Pedestrians: 5th percentile winter coats	.03

* *The reflectance of a surface is defined as the ratio of the reflected flux to the incident flux, i.e. the proportion of the light falling on a surface that is reflected back from the surface in a given direction. A perfectly reflective surface would have a reflectance of 1.0 or 100%.*

TABLE D-5. Visibility Distances and Safety Factors Considering Glare.

Target	Distances (feet)		Safety Factors	
	No Glare	Glare	No Glare	Glare
Large Objects	46 - 394	181 - 351	0.10 - 0.87	0.40 - 0.78
Small Objects	148 - 476	125 - 374	0.33 - 1.06	0.27 - 0.83

TABLE D-6. Expected Response Distances (in feet) to Pedestrian Targets.

Headlamp Beam	Target Location	Clothing	Average Response Distance (ft)	Range
Low	Right	Dark	80	0 - 160
		Light	160	80 - 240
	Left	Dark	60	0 - 120
		Light	120	50 - 200
High	Right	Dark	120	60 - 300
		Light	240	160 - 400
	Left	Dark	120	60 - 300
		Light	240	160 - 400

TABLE D-7. Visibility Distances Considering Reflectance and Intensity.

Intensity (CD)	Visibility Distances (feet)		
	Object Reflectance		
	2%	7%	14%
25,000	180	320	390
50,000	240	400	470
75,000	260	430	520
100,000	280	470	560
125,000	300	500	600
150,000	310	530	650

A very early psychophysical study related visibility of objects of varying reflectance to candelas incident upon them (*39*). Objects were visible at distances as a function of headlamp intensity. Table D-7 shows distances at which the objects were just visible. A study of Table D-6 reveals that a point of diminishing returns is quickly reached when attempting to improve visibility distances by using brighter headlamps. The inverse square law makes it necessary to increase intensity by at least fourfold to obtain double the distance.

Also, effective contrast between object and background (the pavement) tends to diminish with increasing distance between the source and the observer because relative to the distance to the source, the object and background are closer together. Considering the required stopping sight distances at highway speeds, atmospheric scatter of the illumination from the headlamps also plays a significant role by acting to reduce effective contrast for the observer as well as to diminish beam intensity. Recent trends in lowering headlamps for improved aerodynamics, style, reduced car weights to help reach Clean Air and Fuel Efficiency (CAFE) standards (thus making aiming much more susceptible to loading), and lower driver eye heights all interact to shorten headlamp visibility distances.

SUMMARY

These visibility studies have identified some important concepts that will be applied to the critical objects chosen from the accident data. Rae suggested that a suprathreshold visual performance be used to measure driver performance on the roadway because it deals with reactions beyond detection. Keck explained that the drivers' visual perception must adapt to the surroundings and roadway conditions before they can detect an object.

McLean supported the conclusion that the driver needs five minutes of arc to perceive an object on the roadway considering the visibility conditions. He extended this conclusion to state that at 65 meters, 100 millimeters of the object must be visible, and at 130 meters, 200 millimeters of the object must be visible for the driver to detect it. For distances longer than 130 meters and speeds above 90 kilometers per hour the stopping sight distance is beyond human visual capabilities.

Ketvirtis contradicted this last assumption made by McLean. Picha found that the 85th percentile detection distance under ideal conditions ranged from zero meters to 275 meters (903 feet), depending on the height. The 85th percentile recognition distances for the object were much shorter; they ranged from zero feet to 134 feet (41 meters).

Ketvirtis presented results for roadway luminance levels which enabled the driver to see an object at 160 meters (525 feet). If the driver could not see the object at this distance, as suggested by McLean, then no amount of light would make it visible. Ketvirtis also determined that very few drivers elected to come to a complete stop when faced with a hazard in the roadway. Most drivers choose to go around objects larger than 152 millimeters (six inches).

METHODOLOGY

This research incorporated two different, but similar, field studies to determine driver visual capabilities in object detection and recognition. The experimental study design measured drivers' capabilities in detecting and recognizing different sized objects in the roadway, under different lighting conditions. The first study, Study 1, measured driver visual capabilities during daylight conditions. Six objects of various sizes were evaluated in this study. The second study, Study 2, measured driver visual capabilities under nighttime conditions. A completely different set of eight objects of various sizes were evaluated. Each of the studies are further described in this section.

STUDY 1– DAYLIGHT VISUAL CAPABILITY STUDY

Study 1 was conducted at the Texas A&M University Riverside Campus on a closed-course test track and involved a range of driver ages and a variety of different objects. The study was conducted only during the daytime and only between the hours of 10:00 A.M. and 5:00 P.M. in order to achieve maximum sunlight conditions. Dry pavement conditions were also a requirement. There were no horizontal or vertical curves involved with the detection and recognition of the objects. Each object was placed only on the tangent section of the course. Weather conditions were noted each day, including temperature, humidity, and cloud conditions.

These conditions eliminated some of the variables in the study, such as horizontal or vertical alignment, a wet pavement, and night-vision requirements. The conditions presented here are ideal and represent a best-case scenario. Any conditions other than these would only result in shorter distances of object detection and recognition. Objects used, driver performance measures, controlled testing procedure, follow up questions, and course layout are described in the following sections.

Objects Used

Six different sized objects, with varying contrasts, were used for Study 1. The objects ranged from 4 to 18 inches in height. Two objects of different contrast, both six inches in height, were tested because of their analogy to AASHTO's 150-mm object height used for stopping sight distance design purposes. The objects used in Study 1, their dimensions, and their contrast relative to portland cement concrete pavement are presented in Table D-8. Figures D-4 to D-9 are photographs of the objects.

The objects that were used in Study 1 were based on research by Kahl (*40*) in a study of accident reports and accident databases in two regions of the United States. Kahl's research involved determining objects that were probable causes of accidents on various highway classifications.

All objects used remained stationary in the roadway during the study. A particular item of study are the two six-inch objects of different contrasts. The 152 mm (six-inch), stationary object is a particular interest because this is the criterion used in stopping sight distance calculations. The objects were evaluated on their ability to be detected and recognized.

TABLE D-8. Dimensions and Contrasts of Objects Used in Study 1.

Object	Dimensions (millimeters)		Color	Contrast
	Height	Width		
2' x 4' Wood	102	610	Light Brown	Low
Black Dog	152	152	Black	High
White Dog	152	152	White	Low
Tire Tread	203	457	Black	High
Tree Limb	305	457	Green/Brown	Low
Bale of Hay	457	610	Light Brown	Low

FIGURE D-4. Daytime Test Object 1: Two 1 x 4's.

FIGURE D-7. Daytime Test Object 4: Tire.

FIGURE D-5. Daytime Test Object 2: Black Dog.

FIGURE D-8. Daytime Test Object 5: Tree Limb.

FIGURE D-6. Daytime Test Object 3: White Dog.

FIGURE D-9. Daytime Test Object 6: Haybale.

Driver Performance Measure

With the aid of a distance-measuring instrument (DMI) in the test vehicle, distance, time, and velocity were observed at points of object detection and object recognition. With this information, the distance to the object could be calculated to determine if the object was detected and recognized within the minimum stopping sight distance values. The subject's braking and steering behaviors were also observed for each object. This observation showed whether or not the subject showed extreme caution to a particular object; namely to allow a comparison between representative animate objects and inanimate objects, size of the objects, and the perceived hazard of the objects.

Controlled Testing Procedure

Subject Pool. From a desired subject pool of previous study participants, 45 licensed drivers, 15 in each of three different age groups (<25, 25-55, and >55), were contacted and scheduled to drive the course. Each subject was asked personal questions, including their age, the average number of miles they drive in a year, how long they have been legally driving a vehicle, whether or not they had a corrective lens restriction on their driver's license, and if so, if they were wearing their corrective lenses. This check would ensure that all subjects met the legal requirement of a least 20/40 static visual acuity, which is required in Texas. Demographic information also included gender and ethnic background.

Subject Instructions. Each subject, prior to beginning the course, was instructed to become familiar with the Texas Transportation Institute vehicle, a 1991 Ford Crown Victoria with an automatic transmission. By familiarizing the subject with the mirrors, brakes, air-conditioning, seating, and the overall experience of driving an unfamiliar vehicle, he/she would be more relaxed as if the vehicle was a personal vehicle being driven. The subject was then briefed on what the study entailed and why the study was being conducted. Each subject was given the following instructions : "the course is setup to represent a two-lane rural highway, follow the signs as you normally would on this type of road, you will be the only driver on the course at the time of the testing, and the objects will be placed in a location that will not create a hazard for your driving." Each subject was also asked to drive in a safe and usual manner and even though a 55 mile per hour speed limit was posted, the subject was asked to drive at the speed he/she felt comfortable driving.

Test Procedure. Once the instructions had been completed, the subject began driving the course. Each subject was asked to indicate when an object was first detectable by immediately saying "NOW" or in some way indicating that there was something in the road that should not be there. After detecting the object, or if the object was not detected at all, the subject was to then identify the object by verbally indicating what the object is, when it was recognizable.

Each subject drove through the course seven times with an observer in the car. While driving through the coarse each time, one of the objects would be introduced. This change was accomplished by an assistant who would place the object at a predetermined location before the subject would begin each run through the course. As the object was passed by the subject in the vehicle, the assistant remained out of view.

Distance, time, and speed were measured with the DMI by the observer at points of detection and recognition for each object. Distance and time were measured relative to a beginning reference point on the course. Also, even though the subject was instructed that the object would not create a hazard, any change in speed or direction of the vehicle was noted when an object was encountered.

Object Use and Placement. There were four different random orders in which the objects would appear. They are represented in this study as Group 1, Group 2, Group 3, and Group 4. Each subject was placed into one of these four groups. The groups and their order of object appearance are shown in Table D-9. The objects also changed position and location relative to the center-line stripe and the solid-white stripe on the right side shoulder. The objects' locations were at fixed stations so determining detection and recognition distances would be possible. Four consecutive subjects not only saw a different order of objects but different locations at which the objects were encountered. The only objects that remained in the same location for every subject was the tire tread and the bale of hay. These two objects were relatively large and too difficult to relocate to another station.

Follow Up Questions. After driving through the course seven times, the subject was asked additional questions by the observer in the vehicle concerning objects encountered in the road. Each subject was asked if a particular object had ever been encountered while driving a vehicle that created a hazardous situation. A hazardous situation was explained as a "sudden change in speed or direction of the vehicle being operated." If such a situation had occurred, the subject was asked what that object was. Each subject was also asked what the smallest object that he/she considered to be hazardous.

After the observer recorded the verbal responses, the subject was then shown a series of photographs of several objects on a two-lane, rural highway. Table D-10 lists these objects used in the photographs. Based on the environment and how the object looked in the photograph, the subject was asked to rate each object as either a low, moderate, or extreme hazard. A low hazard is explained as "being able to pass over or around the object in the vehicle with very little braking or steering"; a moderate hazard is explained as "being able to pass over or around the object with some degree of difficulty"; and an extreme hazard is explained as "a sudden and drastic change in speed and direction of the vehicle." Each subject was then instructed that the testing was complete. Any questions asked were answered, and any comments made pertaining to the study were recorded.

TABLE D-9. Objects Used In Photographs.

Photograph	Object	Height (inches)
1	Armadillo	6
2	2 ~ 1 x 4's	4
3	2 ~ 1 x 6's	2
4	Wooden Pallet	5
5	Tire Tread	8
6	Tree Limb	12
7	Bale of Hay	18

TABLE D-10. Order of Object Appearance.

Test Run	Group 1	Group 2	Group 3	Group 4
1	2 ~ 1x4's	Bale of Hay	Tree Limb	Tire Tread
2	Black Dog	2 ~ 1x4's	No Object	White Dog
3	Tire Tread	Black Dog	Bale of Hay	Tree Limb
4	White Dog	Tire Tread	2 ~ 1x4's	2 ~ 1x4's
5	Tree Limb	White Dog	Tire Tread	Bale of Hay
6	No Object	Tree Limb	Black Dog	No Object
7	Bale of Hay	No Object	White Dog	Black Dog

Course Layout

The entire length of the concrete pavement course was approximately 5000 feet in length. The course, representative of a two-lane rural highway, began with the longest tangent running north to south being 3000 feet in length, followed by a horizontal curve to the right and another tangent section about 1100 feet in length. This segment was followed by another horizontal curve to the right and a 500 foot tangent section. A layout of the course is shown in Figure D-10. This site was chosen because adequate distance was provided on the 3000 foot straight-aways to introduce objects to the subject. This distance allowed the reduction of driver workload which is required on horizontal curves and allowed the subject to reach a constant speed in the vehicle on the tangent before the object could be detected.

To simulate a two-lane, rural highway, six-inch yellow and white lines were painted and five standard signs were installed on the course, including two 55 mile per hour Speed Limit signs (R2-1) and three Yellow Curve Warning signs (W1-2R) with 35 mile per hour Advisory Speed Plates (W13-1). Pavement markings and sign use and placement meet specifications stated in the Manual on Uniform Traffic Control Devices (MUTCD) (*41*).

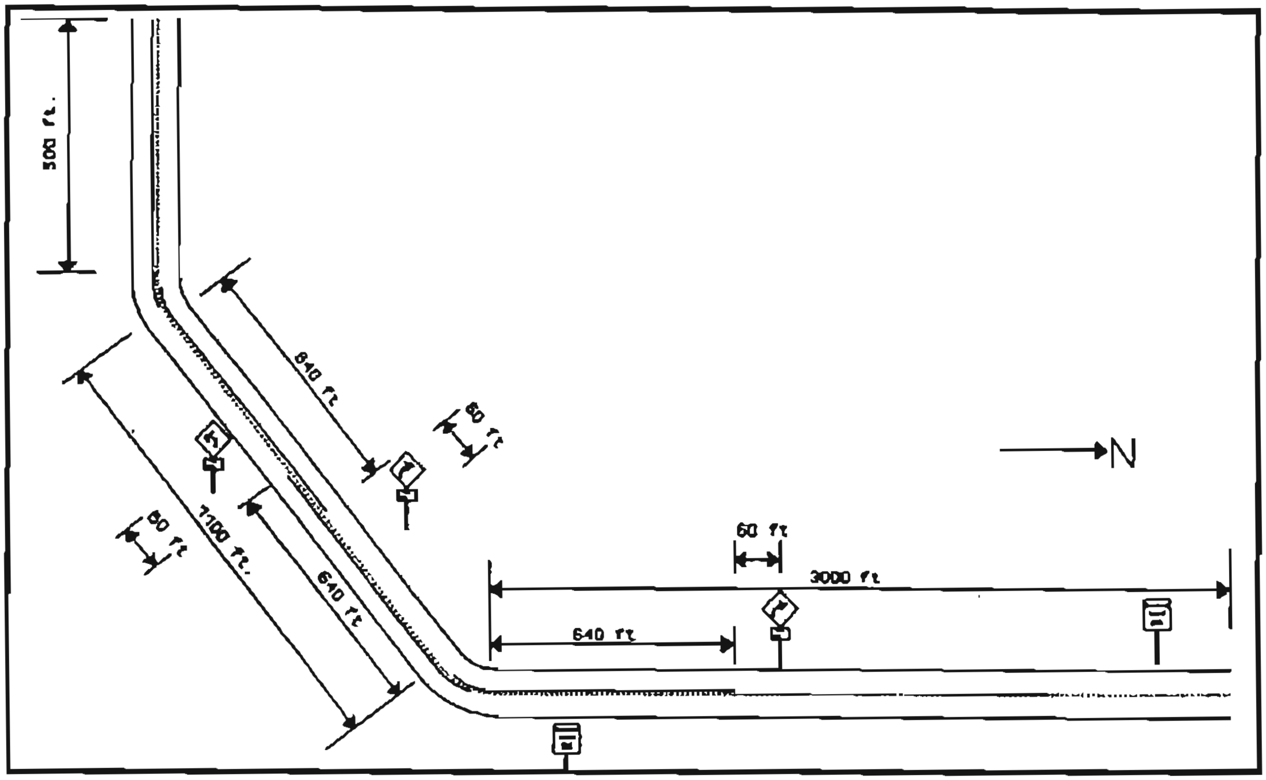

FIGURE D-10. Test Track Layout for Daylight Study.

STUDY 2 – NIGHTTIME VISUAL CAPABILITY STUDY

Study 2 was also conducted at the Texas A&M University Riverside Campus on a closed-course test track. Twenty paid volunteer subjects participated in the study. Ten of the subjects were "younger drivers," from 21 to 25 years of age. Ten of the subjects were "older drivers" from 53 to 71 years. All participants held valid driver licenses. Static, binocular, far vision acuity was determined for each subject with a modified OrthoRater. Snellen acuity equivalents ranged from 20/17 to 20/29 for the young group and from 20/18 to 20/29 for the old group. Acuity was determined, and the experimental task conducted, with the subjects' eyesight corrected as normal for night driving, i.e., if a subject typically wore glasses or contact lenses when driving, he or she also wore them for the study.

Objects Used

For each test run, zero, one or two objects were placed at predetermined positions on the tangent sections of the course. Each object was placed in the travel path of the test vehicle such that the object extended about 1 m into the travel lane as measured from the roadway centerline. A total of seven test objects were used. The test objects were selected to provide a variety of contrasts with the nighttime lighting and pavement against which they were viewed, encompass a range of physical sizes, and be representative of objects often encountered on rural roadways. The test objects used were:

1. A passenger car, with lights off, placed perpendicular to the travel lane, seen from the side;
2. A passenger car, with lights off, placed parallel to travel path, seen from the rear;
3. An orange traffic cone;
4. An upright motorcycle, with lights off and without rider, seen from the rear;
5. A deer with body perpendicular to the roadway and head oriented such that one eye is in the subject's line of sight;
6. A curved portion of a truck tire tread; and
7. A pedestrian, represented by a clothed mannequin, seen in profile.

Additional descriptions of the target objects, their physical dimensions and reflectance measurements are provided in Table D-11.

Driver Performance Measure

Target detection and recognition distances were easily calculated by subtracting the distance from the "start" point to the subject's detection and recognition response, as measured by the DMI, from the known distance from the start point to the target object. Two characteristics of the experimental procedure work to produce somewhat conservative, that is, shorter detection and recognition distance estimates.

TABLE D-11. Target Object Descriptions, Dimensions, and Reflectance.

Object	Object Dimensions		Reflectance measured at	Reflectance*
	Height (m)	Width (m)		
1979 Pontiac Grand Am – side view	1.42 (roof line) 0.81 (front feeder)	4.88	Brown side paint Side chrome strip	0.184 0.367
1979 Pontiac Grand Am – rear view	1.42 (roof line)	1.75	Brown paint White license plate Rear Chrome	0.184 0.550 0.654
Traffic Cone	0.48	0.28 (base)	Orange body	0.322
Motorcycle Rear (1985 BMW R80)			White license plate background	0.528
Deer (Decoy)	1.4 (antler) 1.04 (head) 0.97 (back)	1.17	Mid-body	0.170
Tire Tread	0.22	0.75	Middle of tread	0.031
Pedestrian (mannequin)	1.83	0.30	Dark blue pants Dark green shirt Dirty white shoe Skin	0.021 0.035 0.388 0.371

* *Reflectance was derived for each object from comparison of the measured luminance of the objects, under midday sun, with the luminance of a standard gray card (18% reflectance) superimposed on the objects' surface.*

First, the distances determined in this study are based on verbal reports by the subject. It is possible that a non-verbal response (for example, a key press) may have produced longer distances. Error introduced by the requirement for a verbal response may be partially offset by the increased expectancy of the subject that an object would be encountered somewhere along the test course. Also, because subjects in this study were not driving, they were unburdened of control and guidance tasks normally required by the driving task.

Second, the distances measured are influenced by a small delay due to the reaction time of experimenter operating the DMI in response to subject's verbal signals. The experimenter, however, was well practiced in responding quickly to subject's verbalizations prior to actual data collection. Assuming a lag of 0.1 second between a subject's response and DMI activation, this error, at 56 kilometers per hour (15.5 meters per second) would be less than 2 meters.

Controlled Testing Procedure

Subject Pool. Twenty paid volunteer subjects participated in the study. The ten subjects classified as "younger drivers" ranged from 21 to 25 years of age (mean = 23.1, std. dev. = 1.4). Ten "older drivers" encompassed a range of 53 to 71 years (mean = 65.1, std. dev. = 3.2). All participants held valid driver licenses. Static, binocular, far vision acuity was determined for each subject with a modified OrthoRater. Snellen acuity equivalents ranged from 20/17 to 20/29 for the young group and from 20/18 to 20/29 for the old group. Acuity was determined, and the experimental task conducted, with the subjects' eyesight corrected as normal for night driving, i.e., if a subject typically wore glasses or contact lenses when driving, he or she also wore them for the study.

Subject Instructions. Upon arrival at the study briefing room, participants were given an explanation of the purpose of the study ("to help determine how well drivers can see potential hazards in or near the roadway") and their task, provided with an informed consent document, administered simple acuity and stereopsis tests, and queried about their driving (e.g., approximate annual mileage, any legal or self-imposed driving restrictions, use of corrective lenses, etc).

With the subject in the front passenger seat and the experimenter in the rear, a test technician drove the test car to the beginning of the test course. On the way, the instructions were paraphrased for the subject and any questions answered. On arrival at the test site, a practice drive through the course, once in both directions, was initiated to familiarize the subject with the general layout of the course with no "potential hazards", i.e., target objects, present. The practice runs and all subsequent test runs were conducted at a nominal 56 kph. The subject was informed that a very obvious hazard would be present at the end of the second practice run. The practice object was a highly reflective hazard triangle. The subject was instructed to respond to this object as he or she would to the actual test objects i.e., to announce as soon as an object was detected, and then announce as quickly as possible what that object is. If the subject performed the practice task correctly (as was nearly always the case), the actual testing began. If necessary, additional instruction/clarification was provided.

Test Procedure. The drive to the test site, practice runs and reiteration of subject instructions assured that each subject received at least 15 minutes to adapt to the dark conditions of the test course before testing began. Except for the LED display of the DMI (shielded from the subject in the back seat) and the normal light emitted from the dash instrument cluster, no lights were on in the test car. Each participant was first subjected to six test runs with low beam head-lighting, three in each direction on the course.

On three of the runs, a single target object was placed in one of the three tangent sections of the course. On two runs, an object was placed on two of the tangents, and on one run no objects were on the course. The order in which single-object, no-object and two-object runs were presented and the pairing of objects for the two-object runs, were randomized across subjects. Objects for a given run were placed surreptitiously by test confederates at the end of the preceding test run while the test vehicle faced away from the course. Radio communication between the confederates and in-vehicle experimenters verified the readiness of the course for a run. To preclude alerting subjects if or where a particular may be located, all references to objects and where they were placed was accomplished by pre-arranged run numbers only.

Following completion of the low beam test runs, participants were subjected to an additional six runs with high beam illumination. The same procedures were employed as for the low beam runs except that a different order of object presentation was used. It should be noted that the same seven objects were used for low and high beams. Subjects experienced each object only once under the both headlight conditions. Thus, for the low beam runs, each target object was novel and unpredictable. For the high beam condition, subjects had previously been exposed to each object once before with low beam illumination. The implications of this learning effect for the high beam detection and recognition distances are addressed in the discussion of the results.

Course Layout

An approximately 1.5 kilometer test course, simulating a two-lane, unlighted rural roadway, was established on a concrete taxiway at the Proving Grounds located on the Texas A&M University Riverside Campus. The course is comprised of three tangent sections, approximately 620, 330 and 440 meters in length, connected by two horizontal curves. The test course has a yellow broken centerline and white edge lines. One or two route markers are mounted, in compliance with the MUTCD, on signposts in both directions of travel in each tangent test section. For each test run, zero, one or two objects were placed at predetermined positions on the tangent sections of the course. Each object was placed in the travel path of the test vehicle such that the object extended about one meter into the travel lane as measured from the roadway centerline. A total of seven test objects were used. The test objects were selected to provide a variety of contrasts with the nighttime lighting and pavement against which they were viewed, encompass a range of physical sizes, and be representative of objects often encountered on rural roadways. The test objects used were:

1. a vehicle, with lights off, placed perpendicular to the travel lane, seen from the side,
2. a vehicle, with lights off, placed parallel to travel path, seen from the rear,
3. an orange traffic cone,
4. an upright motorcycle, with lights off and without rider, seen from the rear,
5. a deer with body perpendicular to the roadway and head oriented such that one eye is in the subject's line of sight,
6. a curved portion of a truck tire tread, and
7. a pedestrian, represented by a clothed mannequin, seen in profile.

Additional descriptions of the target objects, their physical dimensions and reflectance measurements are provided in Table D-2. Daylight photographs of each object are included as Figures D-11 to D-17.

The test vehicle, a 1991 Ford Crown Victoria, was equipped with a distance measuring instrument (DMI) operated by an experimenter positioned in the back seat. The DMI provides for measurement between any two points by means of a single switch activation at the start and end of the desired distance. High and low beams of the test car original manufacturer's equipment headlights were aligned in conformance with SAE recommended practice.

RESULTS

This section documents the findings of the driver visual capability studies described in section three. The findings are separated by study -- daylight visual capability study and nighttime visual capability study. The daylight study is subdivided by demographics and personal data, object detection and recognition distances, and hazard rankings; and the nighttime study is subdivided by detection and recognition distances for all drivers and detection and recognition distances by driver age. The final section presents a summary of the important findings from the driver visual capability studies.

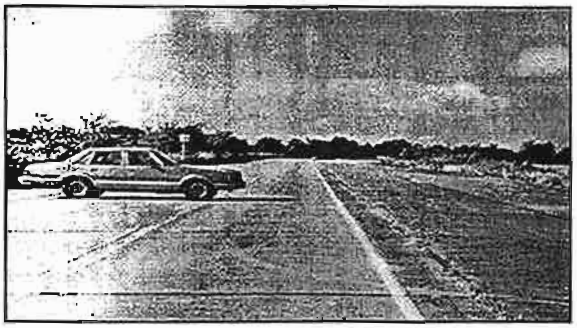

FIGURE D-11. Nighttime Test Object 1: Side of Vehicle.

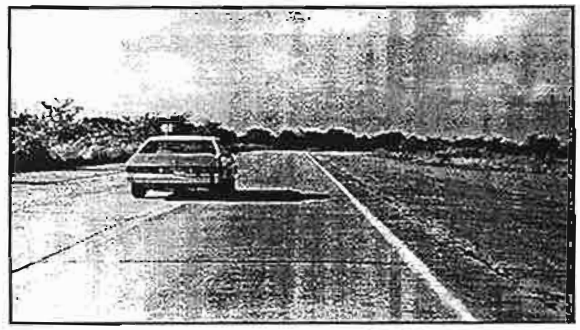

FIGURE D-12. Nighttime Test Object 2: Rear of Vehicle.

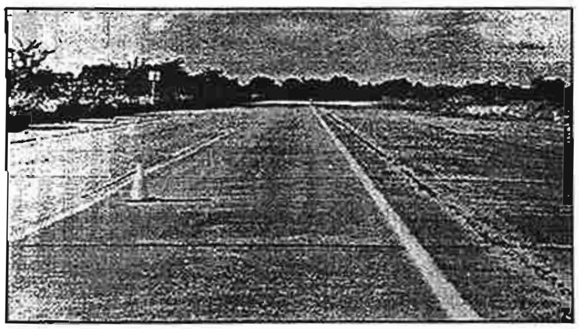

FIGURE D-13. Nighttime Test Object 3: Traffic Cone.

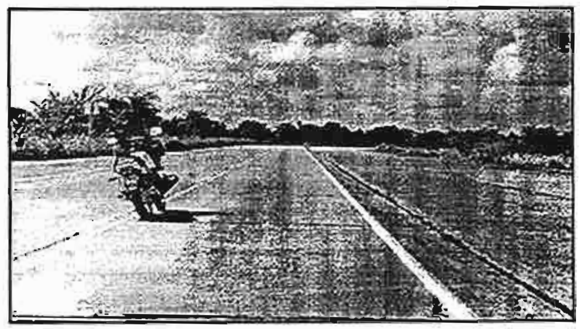

FIGURE D-14. Nighttime Test Object 4: Motorcycle.

FIGURE D-15. Nighttime Test Object 5: Deer.

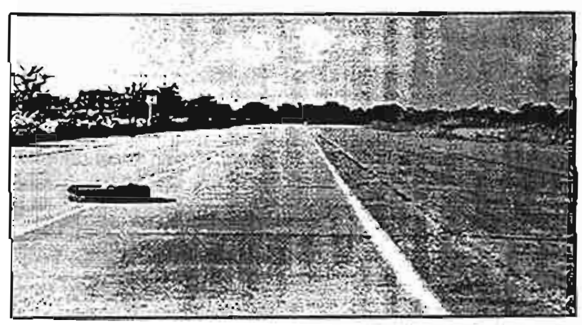

FIGURE D-16. Nighttime Test Object 6: Tire Tread.

FIGURE 17. Nighttime Test Object: Pedestrian Mannequin.

STUDY 1 – DAYLIGHT VISUAL CAPABILITY STUDY

The data that were primarily addressed in this study was the distances of detection and recognition for each object, the hazard rating for the objects shown in the photographs, and a breakdown of the demographic data. The field data were reduced and the distances of detection and recognition and the hazard ratings were calculated.

Demographic and Personal Data

The demographic and personal data addressed in this study included age, gender, ethnic background, corrective lens requirements, and the average number of miles driven in a year. Ethnic background information was collected solely for informative purposes and is not presented here. A listing of pertinent information is presented in Table D-11. Of the total 43 subjects participating, 16 were less than the age of 25, 15 were between the ages of 25 and 55, and 12 were over the age of 55.

All subjects required by law to wear corrective lenses did in fact wear them. A larger percentage (50 percent), however, of subjects over the age of 55 wore corrective lenses, versus 31 percent and 27 percent for less than 25 years of age and between 25 and 55 years of age, respectively. Subjects in the middle age group, on the average, drove 16,000 miles a year. This was the highest of the three age groups. Subjects in the younger age group drove an average of 14,800 miles a year and subjects in the older age group drove an average of 10,000 miles a year. These averages include both genders. Table D-11 provides a breakdown of average miles driven per year for each age group and gender.

Object Detection and Recognition Distances

Results of drivers in all three age groups are presented in Figure D-18 and Table D-12. Represented in Table D-12 are the 15th and 50th percentile values of detection and recognition distances for the six objects studied. The 15th percentile values for detection and recognition of the six-inch black dog was 593 feet and 18 feet, respectively, and the 15th percentile values for detection and recognition of the six-inch white dog was 231 feet and 2 feet, respectively.

The *Green Book* states that an object must be perceived as a hazard before reacting to it. This definition would indicate that stopping sight distance is between the detection and recognition range. The driver definitely has to detect an object but does not necessarily have to recognize it to make a decision to stop. The 15th percentile driver detected the objects in this study, except for the four-inch wooden object and the six-inch white dog, at or above 450 feet.

This distance is the minimum, or optimum, required stopping sight distance for a vehicle traveling between 48 to 55 miles per hour and also a rounded design value designated in the *Green Book*. The values for the six-inch objects differ because of contrast. The black dog was a high contrast object on the concrete pavement and was more easily detectable than a low-contrast white object. Both 15th percentile values, however, for recognition were nearly identical for the 6-inch objects : 18 feet for the black dog and 2 feet for the white dog.

TABLE D-11. Demographic and Personal Data of Participating Subjects.

Age	Gender	# Drivers	Wearing Corr. Lenses	Average Miles Driven Per Year		
				<10,000	10-20,000	20-30,000
<25	Male	10	2	3	4	3
	Female	6	3	3	2	1
25-55	Male	6	1	0	3	3
	Female	9	3	3	5	1
>55	Male	4	1	2	0	1
	Female	8	5	4	3	1
Totals		43	15	15	17	10

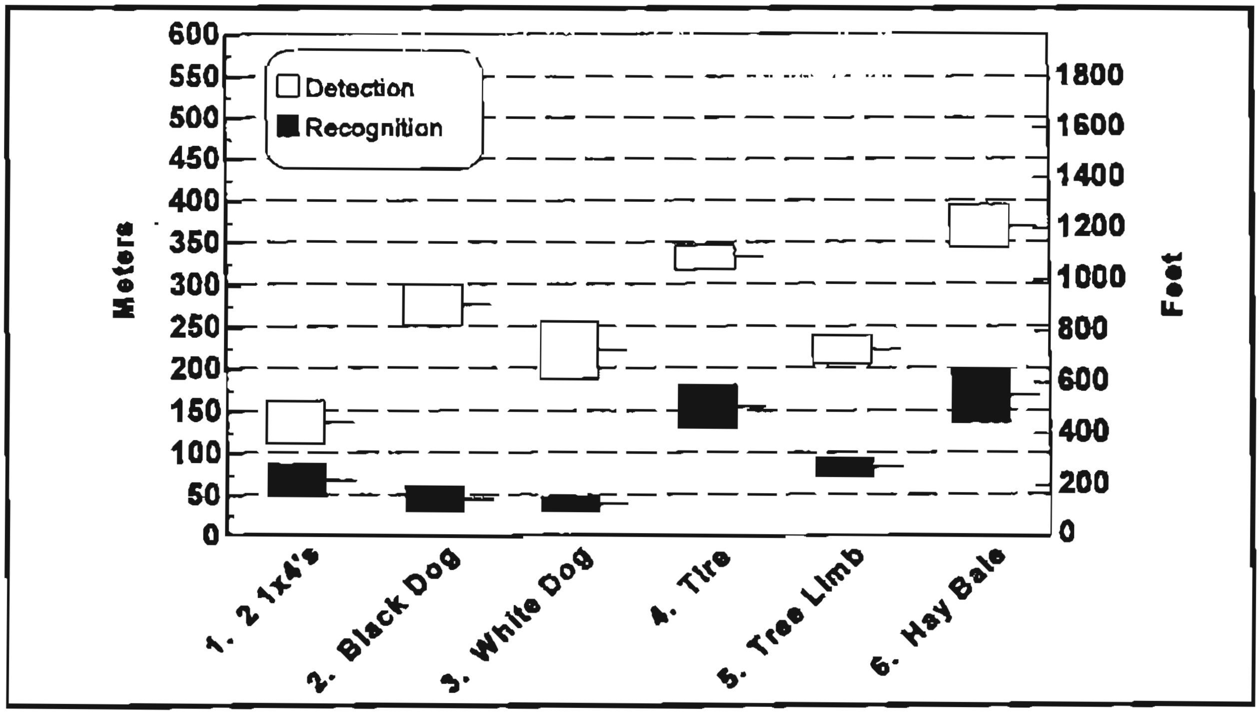

FIGURE D-18. Mean Detection and Recognition Distances With 95 Percent Confidence Intervals.

TABLE D-12. Object Detection and Recognition Distances.*

	15th Percentile		50th Percentile	
Object	Detect	Recognize	Detect	Recognize
2 ~ 1x4's	0	0	113	44
Black Dog	181	5	277	39
White Dog	70	1	213	30
Tire Tread	275	40	333	155
Tree Limb	253	22	218	81
Bale of Hay	254	41	371	169

* *All distances given in meters.*

Hazard Rating

For the photographs shown to the driver, mean responses are depicted in Figure D-19. These responses are similar to what was obtained in a similar study (*34*) which showed a general increase in hazard rating as a function of object size. These objects, however, were not evaluated solely on size, but on appearance, depth, and whether it represented an animate or inanimate object. The 305-millimeter (12-inch) tree limb had the lowest rating of the seven objects. Reasons for this might be that the drivers felt that the tree limb would pose no danger to them or would not damage the vehicle if the object was struck. A tree limb of a different size probably would make a difference in the responses.

The 152 millimeter (6-inch) armadillo shows the second lowest hazard rating, but this may be a result of lack of driver sympathy toward an animal, such as an armadillo. A different representation of an animate object might show more sympathetic responses, meaning more evasive action required in the vehicle to avoid the animal, increasing the hazard rating. The wooden objects show a general increase in hazard rating as a function of size and depth. This increase can be attributed to the driver uncertainty of the vehicle being able to straddle the object. The bale of hay received the highest rating. No other option is available to the driver but to go around this object or come to a complete stop if a collision is to be avoided.

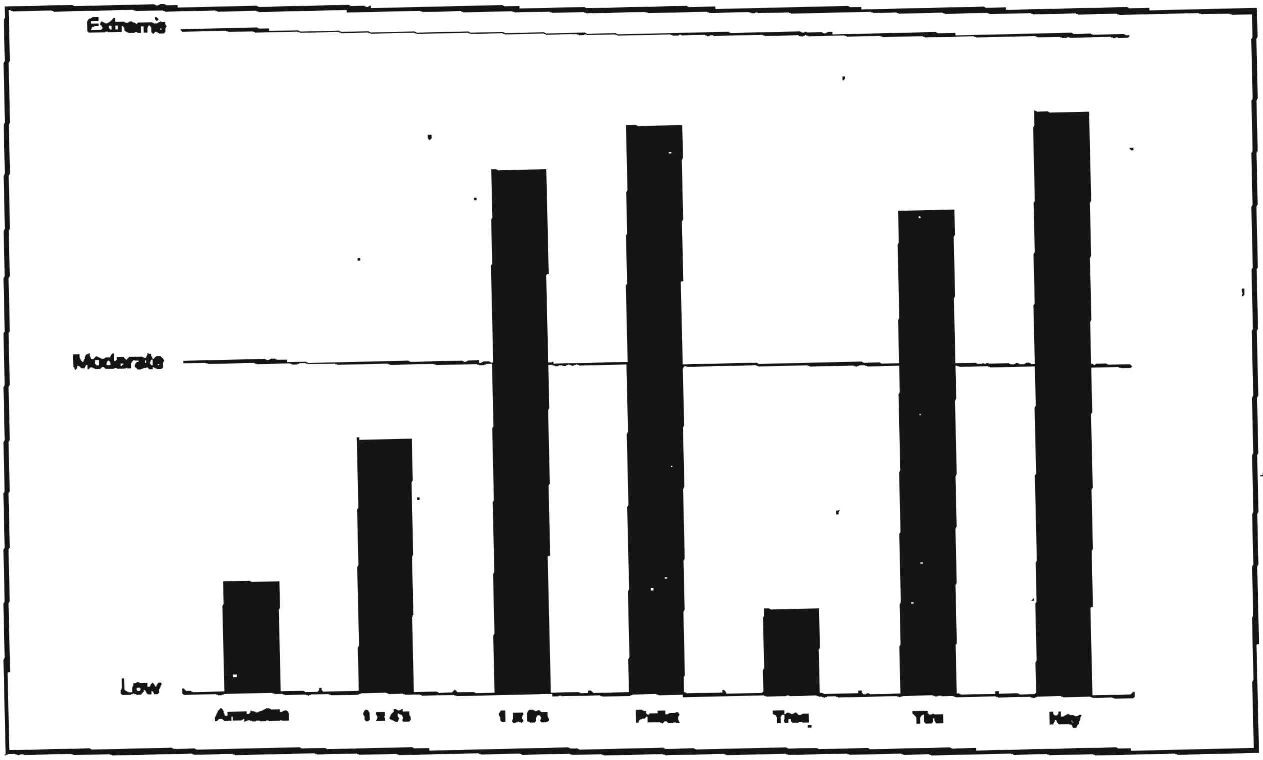

FIGURE D-19. Hazard Rating According To Object.

STUDY 2 – NIGHTTIME VISUAL CAPABILITY STUDY

The data that was primarily addressed in this study was the distances of detection and recognition for each object, the hazard rating for the objects shown in the photographs, and a breakdown of the demographic data. The field data was reduced and the distances of detection and recognition and the hazard ratings were calculated.

Object Detection and Recognition Distances

The observed mean and median detection and recognition distances for the seven objects used in this study are presented in Figures D-20 and D-21 for all subjects, regardless of age, under the low and high beam headlight conditions, respectively. The estimated population 15th percentile distances, assuming normally distributed detection and recognition distances, are also provided.

Figure D-20 indicates that very few drivers can be expected to detect, much less detect and correctly recognize, roadway hazards on dark roadways illuminated only with low beam headlamps at distances as great as the minimum stopping sight distance required under current procedures for roads with a 55 mile per hour design speed (137 meters or 450 feet). Under low beam illumination, the only objects for which the average detection distance, as represented by either the mean or median, exceeded the minimum stopping sight distance for roadways with a 55 mile per hour design speed were the side and rear of the passenger vehicle. The only object that was correctly recognized from more than 137 meters by 50 percent or more of the subjects was the rear view of the car. The object exhibiting the greatest 15th percentile detection distance (116 meters) under low beams was the side view of the passenger vehicle. The longest 15th percentile recognition distance (78 meters) was obtained for the rear of the vehicle. Clearly, both detection and recognition distances under low beam viewing can be expected to be well short of minimum stopping sight distances for a majority of drivers, at least for the types of objects used here.

As expected, objects were both detected and recognized from substantially greater distances with high beam illumination. Except for the tire tread and the traffic cone, the average detection distance exceeded 137 meters (450 feet) for all test objects. Even with high beams, however, only the vehicle, as viewed from the side and rear, and the deer produced average recognition distances greater than 137 meters. The passenger vehicle, from both perspectives, was the only object for which the 15th percentile detection and recognition distances exceeded 137m. High beam detection and recognition distances for all subjects are summarized in Figure D-21. Because all subjects were exposed to the test objects with low beam illumination prior to exposure with high beams, the detection and recognition distances observed with high beams are likely greater than would have been observed had the drivers not seen the same objects before. This prior experience would be expected to have a greater effect on the recognition distances than on simple detection of the objects.

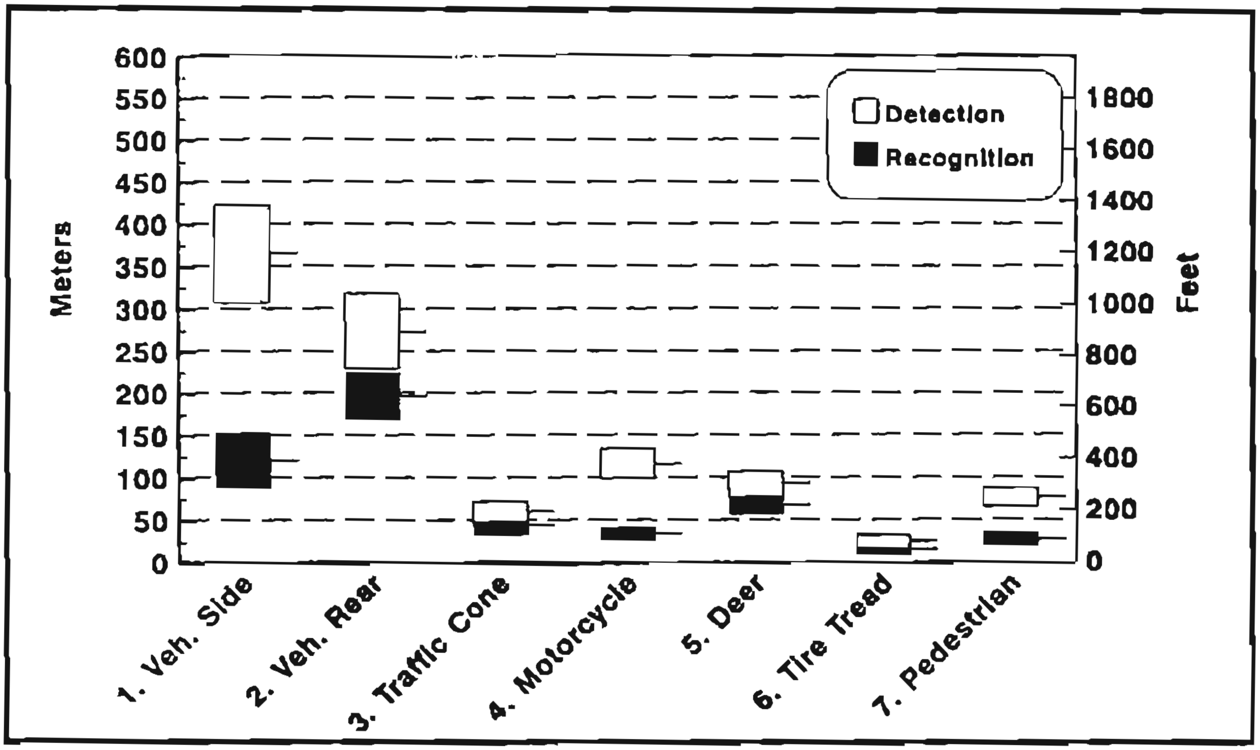

FIGURE D-20. Low Beam Detection and Recognition Distances for All Subjects.

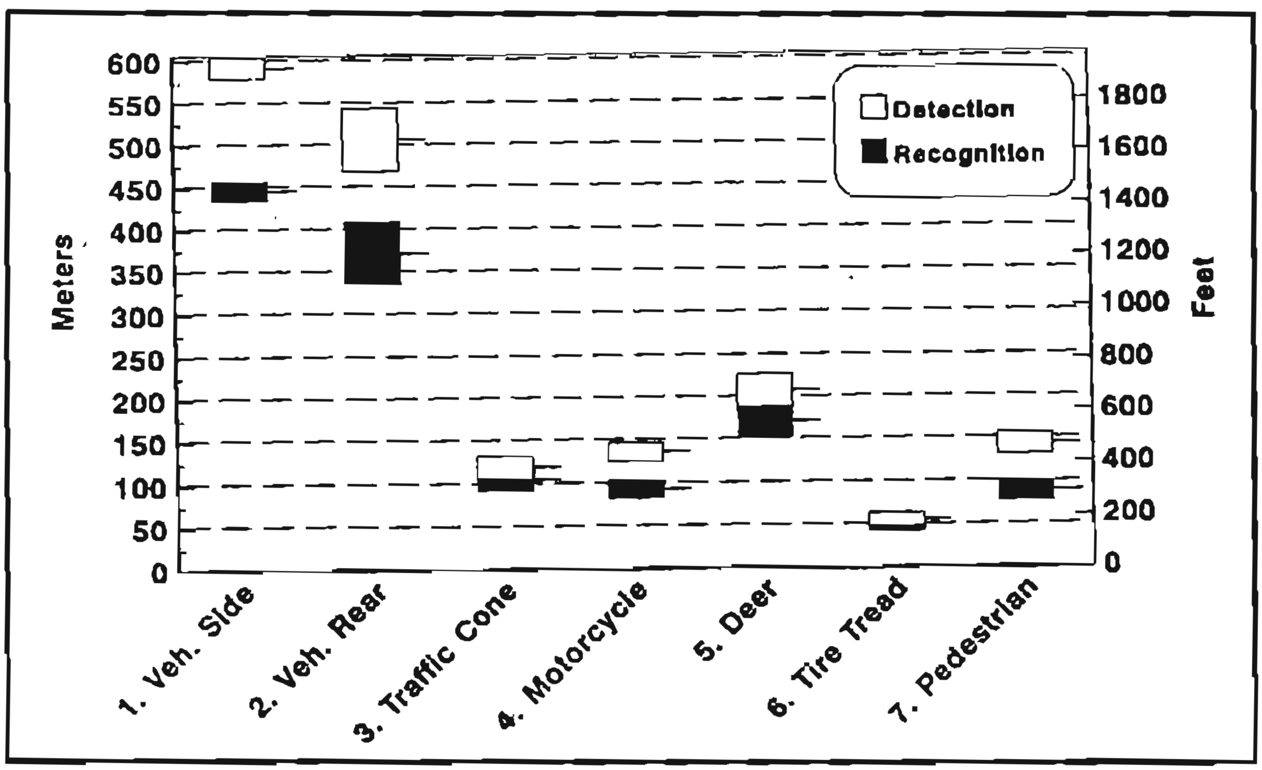

FIGURE D-21. High Beam Detection and Recognition Distances for All Subjects.

Detection and Recognition Distances as a Function of Age

Comparisons of the detection and recognition distances for the younger and older subject groups are illustrated in Figures D-22 and D-23 under low beam and high beam illumination, respectively. Mean low beam detection and recognition distances for younger and older drivers are shown in Figure D-22. With two exceptions, the sample mean detection and recognition distances for all objects were longer for the younger subjects than the older participants. The exceptions, both under low beam illumination, were detection distances for the vehicle side and the motorcycle. For these objects, the observed average detection distance was greater for the older group. T tests performed on the 28 comparisons between the average detection and recognition distances of young and older drivers reveal that in only eight cases were the differences statistically significant ($p < .05$). The difference between younger and older subjects on the two objects for which older drivers produced longer average detection distances was not statistically significant. As indicated by the asterisks in the figure, differences between detection distances for young and old observers were significant only for the deer and tire tread. The difference in recognition distance between younger and older drivers also was statistically significant for the rear of the vehicle.

Mean high beam detection and recognition distances for younger and older drivers are shown in Figure D-23. Somewhat greater differences between the two age groups were evident when the objects were illuminated with high beams. While differences in detection distances were significantly longer for the young group only for the motorcycle, average recognition distances were statistically significantly longer for the young group for four of the seven objects tested: the vehicle side, vehicle rear, motorcycle, and pedestrian.

With the two exceptions noted previously, the sample means for the younger group (both detection and recognition) were consistently longer than for the older group. The combination of a relatively small sample size and very high variability between subjects may account for the lack of statistical significance.

SUMMARY

The findings indicate that drivers can detect a high contrast 6-inch object within the minimum parameters established by AASHTO for a driver traveling at 55 miles per hour. The same can be said for objects that are greater than 6-inches in height, regardless of contrast. Further evidence however, suggests that drivers do not have the visual capabilities to recognize high or low contrast objects that are less than 12 inches in height within AASHTO parameters. Recognition is not totally necessary for stopping sight distance, but the driver must be able to recognize the object as a hazard.

The results of this study suggest that under nighttime conditions where illumination of potential roadway hazards is provided only by the vehicle headlamps, a substantial proportion of the driving population will likely not be able to detect or recognize a wide variety of hazardous objects on the roadway at the distance currently prescribed for daytime stopping sight distance on rural highways when traveling at 55 miles per hour. The only exception is another vehicle.

Detection and, more especially, recognition of potentially hazardous objects at distances comparable to currently defined stopping sight distance is particularly unlikely when low beam headlights are selected.

CONCLUSIONS AND RECOMMENDATIONS

The research documented in this paper examined driver visual capabilities relative to detection and recognition of stationary objects in the roadway. The effects of headlights on a driver's nighttime visual capabilities are also documented. The literature was reviewed to document what had been done in the past, and the current state of the practice. Field data were collected to verify past findings and support recommended values. Conclusions and recommendations of this study are discussed in the following sections.

CONCLUSIONS

- The literature review revealed that the 150-mm object used in the AASHTO stopping sight distance model is not based on whether such an object can be detected and recognized as a hazard by drivers. Rather, the recommended object height is based on some combination of hazard potential and safe, aesthetic, and cost-effective design.

- The literature review also suggested that under nighttime conditions and for unlit objects, driver visual and vehicle headlight capabilities combine to limit detection and recognition of small and intermediate sized objects to approximately 100 meters.

- Under ideal conditions (daylight and straight, flat roadways), most drivers can detect most small objects (small low contrast objects are the exception) at or beyond minimum AASHTO stopping distances for rural highways (140 meters); however, with regard to their potential hazard, few drivers can recognize small objects at these distances.

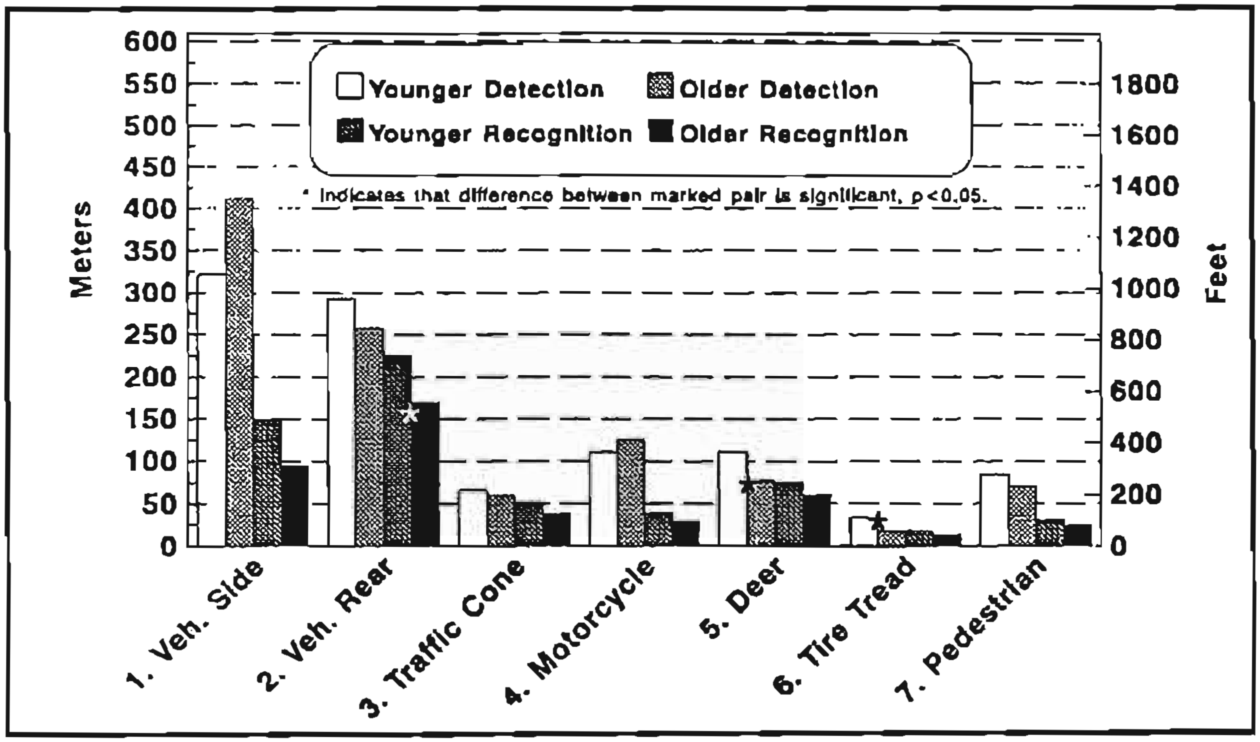

FIGURE D-22. Low Beam Detection and Recognition Distance by Driver Age.

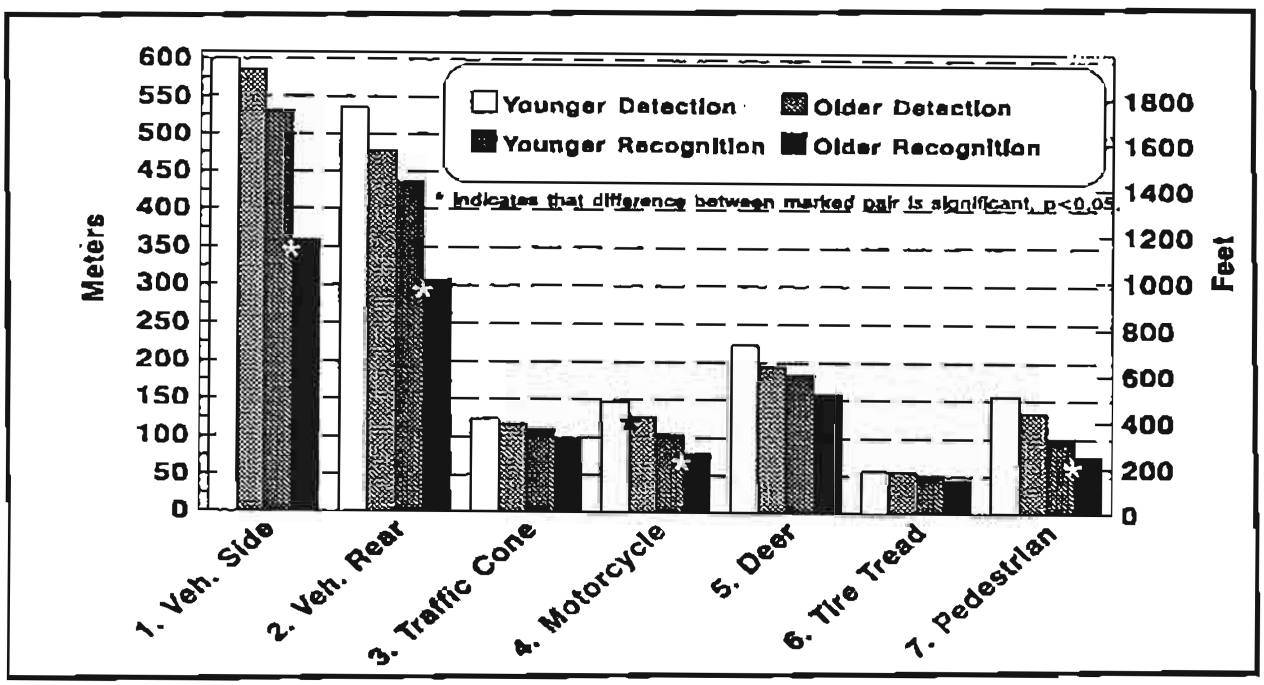

FIGURE D-23. High Beam Detection and Recognition Distances by Driver Age.

- Under low beam headlamps on straight, flat roadways, very few drivers can detect objects as large as deer, motorcycles, or pedestrians at or beyond minimum AASHTO stopping sight distances for rural highways (140 meters). High beam headlights increase detection of these objects to near the minimum AASHTO stopping sight distances for rural highways; however, without retroreflectivity or illumination, most smaller objects are not detectable at these distances.

  - Under low beam headlights and straight, flat roadways, almost all drivers can detect and recognize parked and/or disabled vehicles well beyond minimum AASHTO stopping sight distances for rural highways. The illumination and retroreflection of the vehicle's taillights and/or side reflectors are the reason for the increased visibility of these objects.

RECOMMENDATIONS

- The object selected for the AASHTO stopping sight distance model should represent a realistic hazard and be within the driver's visual capabilities. The taillights of a vehicle in the roadway ahead satisfy both criteria. Smaller objects are beyond a driver's visual capabilities at night and may not represent a realistic hazard to most drivers.

- The object height selected should consider the minimum legal taillight height, the distribution of taillight heights in the current and/or future vehicle fleet, and the concern of not producing overly short vertical curve lengths. That is, minimum vertical curve lengths should consider appearance, comfort, and safety.

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APPENDIX E -

DRIVER EYE AND VEHICLE HEIGHTS

INTRODUCTION

The minimum length of horizontal and vertical curves is controlled by three parameters: required stopping sight distance (sum of perception-reaction and braking distances), driver eye height, and object height. This minimum length of curve provides for at least the minimum stopping sight distance along the entire length of the curve. AASHTO's driver eye height, one of the three parameters, has decreased from 5.5 feet (1676 mm) in 1940 (*1*) to 1070 mm (3.51 feet) in 1994 (*2*), and at least one study (*3*) has recommended even lower values as appropriate for design. Because this parameter directly affects vertical curve lengths, there is a need to determine the appropriate driver eye height for use in geometric design of highways.

The objective of this study was to use real-world data to determine appropriate driver eye height values for developing geometric design criteria. A secondary objective of the study was to determine appropriate values for headlight, taillight and vehicle heights for developing geometric design criteria. To accomplish these objectives, the following tasks were conducted:

- Conduct a literature review of driver eye and object height studies;
- Collect real-world data and construct a cumulative distribution of driver eye, headlight, taillight, and vehicle heights as determined by the current vehicle fleet;
- Determine if driver eye, headlight, and taillight height values are decreasing by stratifying the collected data by vehicle age;
- Obtain a cumulative distribution of the 1993 vehicle fleet taillight and headlight heights based on the 1993 sales of each vehicle type; and
- Recommend a value of driver eye, headlight, taillight, and vehicle heights for developing geometric design criteria.

The findings of the *Driver Eye and Vehicle Height* studies are divided into five sections. The first section, *Introduction*, presents the introduction, study objectives, and organization of the appendix. The second section, *Literature Review*, includes a review of the findings from previous driver eye height and object height studies. The third section, *Methodology*, describes the data collection and reduction procedures used to obtain values of driver eye, headlight, taillight, and vehicle heights. The fourth section, *Results*, presents the findings of the data collection and analysis efforts and describes significant findings in narrative form. The fifth section, *Conclusions and Recommendations*, presents conclusions and recommendations for driver eye, headlight, taillight, and vehicle height values for establishing geometric design criteria.

LITERATURE REVIEW

Since the concept was first introduced in the 1920s, several studies have examined driver eye height for establishing the geometric design criteria. At that time, driver eye height was assumed to be 5.5 feet (1676 mm); and with changes in the vehicle fleet over the past 70 years, it has decreased to 1070 mm (*2*), with some studies recommending an even lower value. The decrease in driver eye height over the years can be attributed to changes in vehicle design, most recently in the 1970s and early 1980s when fuel economy became an important issue, and vehicle manufacturers responded to the challenge with smaller and more compact vehicles. To compensate for this change, the design driver eye height was decreased to 3.5 feet (1067 mm) in 1984; however, it is not known whether this downward trend in driver eye height has continued into the 1990s.

In addition to driver eye height, object and headlight heights are important elements of the current procedure for determining horizontal and vertical lengths that provide required stopping sight distances. Object height, like driver eye height, has varied significantly since its inception in the 1920s, when its value was suggested as 5.5 feet (1676 mm) or the same as driver eye height. The object height used in AASHTO's current design standards is 150 mm (*2*) with some studies suggesting lower values so that drivers can pass over the object without damaging their vehicle, and other studies suggesting higher values because drivers cannot see small objects at the distances required for stopping distance at high speeds. An alternative object height that has been suggested by some researchers is the height of a vehicle's taillight.

As a result of these differing points of view, there is a need to resolve the issue of what value of driver eye height to use in developing geometric design standards. There also is a need to investigate vehicle-related parameters, such as headlight heights, taillight heights, and vehicle heights, that are used in the design of highways and streets. This section presents a summary of the driver eye height, object height, and headlight and taillight height studies that are in the literature.

Driver Eye Height Studies

Hall and Turner (*3*) traced the history of stopping sight distance requirements, and in effect traced the history of driver eye and object heights. The results of Hall and Turner's historical investigation are summarized in Table E-1. The sources listed in Table E-1 are text books and state and national geometric design policies. Note that from 1921 to 1990, driver eye heights decreased from 5.5 feet (1676 mm) to 3.5 feet (1067 mm) and object heights decreased from 5.5 feet (1676 mm) to 6 inches (152 mm); however, also note that object heights were 4 inches (102 mm) from 1940 to 1965. The net effect of these changes has been an increase in the required lengths of vertical curves.

There have been several driver eye height studies during the past 50 years, some of which support AASHTO's recommended values and others which support other values. These studies are not included in Table E-1, but they can be separated into two broad categories—analytical studies and empirical studies.

Analytical Studies. One of the earliest and most comprehensive efforts at quantifying driver eye height was done by General Motors. Driver eye heights had been calculated for each model year vehicle from 1936 to 1957 at the General Motors Proving Ground, and a summary of driver eye height for each vehicle and model year was reported by Stonex (*17*) in 1957. Driver eye heights were determined based on the average vehicle seat cushion height and the average seated eye height for a group of male drivers. This latter value was found to be approximately 28.5 inches (724 mm). In addition, it was found that the average 1936 vehicle seat cushion depressed 2 inches (51 mm).

Based on these values, it was determined that the average driver eye height for each vehicle model year fell from 4.75 feet (1445 mm) in 1936 to 4.25 feet (1295 mm) in 1957. Additional investigation by Stonex revealed that the average 1957 seat cushion depressed approximately 4.2 inches (107 mm), reducing the average driver eye height to 4.04 feet (1232 mm). Based on future vehicle height estimates and an assumed minimum vertical clearance dimension between the driver's eye and the top of the vehicle, Stonex predicted that the mean driver eye height would not fall much below 43 inches (1092 mm).

Olson et al. (*18*) recommended a design value of 40 inches (1016 mm) for driver eye height based on an analytical approach, rather than experimental data. The analytical approach involved determining the distance from the ground to a "seating reference point" (a point defined by the driver's seated position) and from the seating reference point to the driver's eye. This latter distance was obtained by using a method recommended by Hammond (*19*) in which an eyellipse is used to determine the vertical and horizontal position of a driver's eye in a vehicle.

A vertical and horizontal component of eye height is needed because a driver's eye height is dependent on both the height of the driver and the position in which that person sits. Plotting a multitude of these points, creates a cluster of points in the general shape of an ellipse, hence the name "eyellipse." The lowest percentile eyellipse value would be represented by a tangent at the lowest point of the eyellipse, whereas the 50th percentile value would be represented by the midpoint of the eyellipse. Other percentiles are determined in the same manner and illustrated in Figure E-1.

A cumulative distribution of the distance from the ground to the seating reference point was obtained for all foreign and domestic vehicle models that comprised a measurable volume of the 1981 U.S. sales market. The two distributions for distance from the seating reference point to the eyellipse and ground were combined to obtain a distribution of driver eye heights. The study concluded that almost 25 percent of the driver eye heights were below the current 3.5 feet (1067 mm) standard and recommended that the design driver eye height be reduced to 40 inches (1016 mm) to encompass 95 percent of the driving population; i.e., 95 percent of the driver eye heights were above 40 inches (1016 mm).

Empirical Studies. A 1960 study by Lee (*20*) investigated the possibility of a reduction in driver eye height from the value that was then being used in design, 4.5 feet (1372 mm). The study involved photographing moving vehicles to capture the driver's profile and vehicle on film. Reference markers were placed on the pavement to aid in converting scaled values from the photo to their corresponding actual value. Of the 2,000 photos taken at three different sites, 761 were used in the driver eye height analysis. The sample of automobiles included 21 U.S. and 18 foreign models, with model years ranging from 1933 to 1959. The study found that more than 95 percent of the driver eye heights were less than the 4.5 feet (1372 mm) design value, 50 percent of the driver eye heights were less than 4.23 feet (1289 mm), and 15 percent of the driver eye heights were less than 3.95 feet (1204 mm). As a result of this study, AASHO recommended a design driver eye height of 3.75 feet (1143 mm) in 1965.

A 1970 study by Glennon (*21*) evaluated the design standards used in the 1965 AASHO policy. The evaluation was based on existing practices of that time and considered the criteria that were employed in developing the standards. The study concluded a 3.75 feet (1143 mm) driver eye height value was "reasonably representative of current production automobiles," but collected no data to support this conclusion. Glennon suggested that the actual driver eye height might be lower than 3.75 feet (1143 mm) because of the large number of small vehicles in the current vehicle fleet. This suggestion was the impetus for several studies in the next two decades investigating possible reductions in driver eye height.

A 1978 study was the first to investigate the possibility of increased numbers of smaller vehicles reducing driver eye heights. Boyd et al. (*22*) collected data on moving vehicles

TABLE E-1. History of Driver Eye and Object Heights as Related to Stopping Sight Distance.

Source & Date	Driver Eye Height	Object Height
Harger, 1921 (*4*)	1676 mm (5.5 ft)	1676 mm (5.5 ft)
Agg, 1924 (*5*)	n/a	~ 1524 mm (5 ft)
Michigan, 1926 (*6*)	n/a	~ 1524 mm (5 ft)
Oregon, 1935 (*7*)	n/a	~ 1524 mm (5 ft)
Wiley, 1935 (*8*)	n/a	~ 1524 mm (5 ft)
Conner, 1937 (*9*)	n/a	~ 1524 mm (5 ft)
Bateman, 1939 (*10*)	1524 mm (5 ft)	~ 1524 mm (5 ft)
Agg, 1940 (*11*)	1524 mm (5 ft)	~ 1524 mm (5 ft)
AASHO, 1940 (*1*)	1372 mm (4.5 ft)	102 mm (4 in)
AASHO, 1954 (*12*)	1372 mm (4.5 ft)	102 mm (4 in)
AASHO, 1965 (*13*)	1143 mm (3.75 ft)	152 mm (6 in)
AASHTO, 1970 (*14*)	1143 mm (3.75 ft)	152 mm (6 in)
AASHTO, 1984 (*15*)	1067 mm (3.5 ft)	152 mm (6 in)
AASHTO, 1990 (*16*)	1067mm (3.5 ft)	152 mm (6 in)
AASHTO, 1994 (*2*)	1070 mm	150 mm

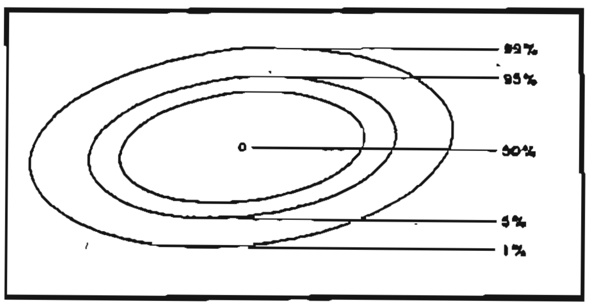


FIGURE E-1. Percentile Tangent Lines of an Eyellipse. (*18*)

in much the same manner as Lee (*20*); however, data were only collected for the 15 most frequently registered domestic models. The 15 domestic vehicles were ranked in descending order based on the number of registered vehicles in the U.S. in 1975, and the number of data points in the database were based on this ranking process. The most frequently registered vehicle was represented by 20 data points, and each of the remaining 14 vehicles was represented by one less data point than the vehicle preceding it in the ranking; i.e., the 5th most frequently registered vehicle was represented by 16 data points in the database and the 15th most frequently registered vehicle was represented six data points in the database. Thus, 195 driver eye heights for the 15 different vehicles (1974-1977 model years) were included in the final database. The average driver eye height was 3.69 feet (1125 mm) and the 15th percentile driver eye height was 3.49 feet (1064 mm).

A 1979 study by Haslegrave (*23*) investigated the driver eye height of two socially different areas in Great Britain using a photographic technique. The technique involved photographing moving vehicles as they entered a parking garage. A profile view of the driver was obtained on film. The study also included an anthropometric survey in which the driver's age, gender, stature, and eye height were obtained. In addition, measurements of the driver's side window and distance to ground were obtained to convert scaled measurements from the photographs to real values of driver eye height. A total of 825 data points was included in the study's database. Results indicated that the average and 15th percentile driver eye heights were 1145 mm (3.76 feet) and 1092 mm (3.58 feet), respectively. An interesting finding from this study was that the lowest eye heights were not necessarily those of short drivers, nor drivers of small cars.

A 1979 U.S. study also investigated the possibility of smaller vehicles causing a decrease in driver eye height. Cunagin and Abrahamson's (*24*) study involved photography of moving vehicles. They placed a camera on the side of a road at a known distance from the lane in which traffic was being photographed. Driver eye height values were determined by measuring distances on the photos and scaling them to corresponding heights in the field. The measured values needed some adjustment because the reference system was not in the same plane as the driver's eye and the data were collected at a site on a slight upgrade. Of the 161 observations that were made at the study site, 62 were small cars, 86 were full size vehicles, and 13 were pick-up trucks. Although the sample size was relatively small, the study concluded that 89 percent of the small vehicles and 73 percent of the full size vehicles had driver eye heights that were less than the design value at that time (3.75 feet). The 15th percentile value was 3.5 feet (1067 mm).

In 1987, Barker (*25*) studied driver eye height at intersections in Australia. He believed that driver eye heights measured at an intersection might be higher than at midblock locations because the driver might be more alert at the intersection, thereby sitting higher than normal. A video camera was placed alongside the approaches to two intersections and stopped vehicles were filmed at both sites. One of the intersections was in a wealthier area of town thought to produce newer vehicles and thus potentially different driver eye heights. A level target with horizontal lines spaced 0.5 meters apart was used to determine a scaling factor. A second video camera was placed parallel to the line of travel to determine the lateral placement of the vehicle and to provide a height adjustment for the first camera. Driver eye heights were obtained for 1124 vehicles, including passenger cars, station wagons, and utility vehicles. The average driver eye height was 1130 mm (3.71 feet) and the 15th percentile value was 1070 mm (3.51 feet). Interestingly, the mean driver eye height was 30 mm lower at the site in the wealthier area.

A summary of the driver eye height studies since 1957 is given in Table E-2. Note that recent studies using data from the field concluded that 85 percent of the driver eye heights exceed 3.5 feet (1067 mm). In contrast, the recent analytical methodology using the "eyellipse" concluded that only 75 percent of the driver eye heights exceed 3.5 feet (1067 mm); however, this procedure assumes independence between the seat height and seated eye height distributions: an assumption that was not supported by Haslegrave's study.

Sensitivity Studies

There is no question that driver eye height has decreased over time; however, there is some debate regarding its relative importance with respect to vertical curve design. A 1982 study by Farber (*26*) related changes in driver eye height to the other parameters used in vertical curve design. Table E-3 provides the equivalent change in speed, friction, reaction time, and object height of a 3-inch (75 mm) 6-inch (150 mm) change in driver eye height. A negative sign indicates that the change is in the opposite direction of driver eye height. The results suggest that a small change in some parameters is equivalent to a larger change in driver eye height. For example, a change in vehicle speed from 60 mph (97 km/h) to 59 mph (95 km/h) reduces stopping distance the same amount that a 3-inch (76 mm) change in driver eye height reduces sight distance.

Farber noted that unless it contrasts strongly with the road surface, a 6-inch (150 mm) object may not be detected for some time after it theoretically can be seen by the driver.

TABLE E-3. Equivalent Changes to Design Parameters as Related to Changes in Driver Eye Height. (*26*)

Parameter	Reference Value	Equivalent Value for Change in Eye Height	
		76 mm (3 in)	152 mm (6 in)
Speed	60 mph (97 kph)	0.9 mph (1.5 kph)	1.8 mph (2.90 kph)
Friction	0.3μ	- 0.011μ	- 0.023μ
Reaction Time	2.5 sec	0.16 sec	0.32 sec
Object Height	6 in (152 mm)	- 1.1 in (- 28 mm)	- 2.4 in (- 61 mm)

He added that it is pointless to require long stopping sight distances if the object cannot be seen at that distance. His recommendations were to investigate the possibility of accident studies or driver visual capabilities limitations for determining appropriate object heights for geometric design. He hypothesized that these studies might indicate that a 12 or 15- inch (305 or 381 mm) object height is more representative of real world hazards that drivers can see and need to avoid.

Gordon et al. (*27*) investigated the driver components of highway design standards and determined which of them may be inadequate for 1984 highway conditions and driver behavior. Highway design standards were reviewed to identify those standards involving driver characteristics, and sensitivity analyses were used to determine if the design standard was responsive to a change in the associated driver characteristic specification. Stopping, passing, intersection, and railroad-highway grade crossing sight distances were recommended for further investigation. Sight distance recommendations did not include a change in driver eye height, as it was found that a 3- inch (76 mm) reduction in eye height would only require a 2.5 percent increase in vertical curve length.

Hall and Turner (*3*) compiled a list of several studies that conducted a sensitivity analysis of driver eye height on stopping sight distance and vertical curve length. They concluded that there was "a strong consensus among researchers that a moderate reduction in driver eye height produces a small change in vertical curve length and stopping sight distance." Most researchers reported 1 to 5 percent changes in curve length and stopping sight distance depending on the assumed reduction in driver eye height. Hall and Turner (*3*) also provided extensive discussion on the sensitivity and effect of object height on vertical curve length. They

TABLE E-2. Summary of Empirical Driver Eye Height Studies Since 1957.

Results/ Parameters	Study					
	Stonex (*17*)	Lee (*20*)	Boyd et al. (*22*)	Cunagin & Abrahamson (*24*)	Haslegrave (*23*)	Barker (25)
Average Driver Eye Height Value	1295 mm (4.25 ft)	1289 mm (4.23 ft)	1125 mm (3.69 ft)	n/a	1145 mm (3.76 ft)	1130 mm (3.71 ft)
15th Percentile Driver Eye Height Value	1242 mm (4.07 ft)	1204 mm (3.95 ft)	1064 mm (3.49 ft)	1067 mm (3.50 ft)	1092 mm (3.58 ft)	1070 mm (3.51 ft)
Vehicle Types*	PC	PC	PC	PC, PU	PC, PU, V	PC, PU
Data Points	1957 Domestics	761	195	1478	825	1124
Data Collection Method	@ GM Proving Grounds	Photography	Photography	Photography	Photography	Video
Year of Study	1957	1960	1978	1979	1979	1987
Country	U.S.	U.S.	U.S.	U.S.	U.K.	Australia

* PC = Passenger Cars PU = Pickup Trucks/Sport Utilities TT = Trailer Tractors V = Vans B = Buses

noted that four of the six studies involving object height sensitivity analysis concluded that vertical curve length is more sensitive to changes in object height than to changes in driver eye height. The authors went on to discuss the implications of maintaining or reducing the current object height:

> "Consider the situation where 600 feet (180 m) of ight distance is required. The current stopping sight distance model assumes that on a tangent level road, the normal driver should not have a problem seeing a 6 in (150 mm) high object at this distance...this corresponds to seeing a 0.2 inches (5 mm) high object at 20 feet (6 m). By comparison, the standard letters on a 20/20 eye chart are 0.35 inches (9 mm) high. Because of variations in driver licensing requirements and the general deterioration of a driver's visual abilities with age, the prudent highway engineer might plan for drivers with a static visual acuity of 20/40. In other words, the design driver must only be able to distinguish among objects that are 3.5 times as large as the object assumed for sight distance purposes. Granted, the driver does not have to read the object. On the other hand, the object need not have the contrast, either with itself or with the roadway, that is provided by a black-and-white eye chart. In addition, the static acuity measured in a standard vision test imposes a less demanding requirement than the dynamic acuity required in the driving task."

Headlight and Taillight Studies

Headlight heights and angle of upward divergence are used to provide required stopping sight distances in the design of sag vertical curves. It is important to note that the driver's visual field of view at night is limited to the illumination of the vehicle's headlights, which has been suggested to be approximately 300 feet (90 m) (*28*); therefore, a low contrast object in the roadway, especially a small 6-inch (150 mm) object, would not be seen at distances beyond 300 feet (90 m) regardless of the available stopping sight distance. A vehicle's taillight, however, could be seen at night at distances greater than that illuminated by the vehicle's headlights. Taillights would alert the driver of a potentially hazardous object in the roadway, namely another vehicle in the same lane, especially at night. Thus, from purely a visibility standpoint, taillight height might be an appropriate object height for use in the design of vertical curves.

Standard 108 of the *Federal Motor Vehicle Standards* (*29*) provides requirements for lighting equipment and its placement on motor vehicles. Taillights and headlights for all motor vehicles must be located on either side of the vertical centerline of the vehicle and as far apart as "practicable." Headlights may be no lower than 22 inches (560 mm) and no higher than 54 inches (1370 mm). Both the upper and lower beam lamp should be at the same height. It should be noted that the minimum headlight height requirements were reduced from 24 inches (610 mm) to 22 inches (560 mm) inches the 1980s. Taillights are required for trailers as well as motor vehicles. Taillights must be mounted no lower than 15 inches (380 mm) and no higher than 72 inches (1830 mm). The point of reference is to be taken as the height above the road surface measured from the center of the item on the vehicle.

Cobb (*30*) measured the headlight height of 452 stopped vehicles at two sites in which a police officer flagged vehicles to the side of the road. Headlight heights were measured manually and a cumulative frequency plot was produced for the four vehicle types in the sample. The average and 15th percentile value for headlight height of the four vehicle types are presented in Table E-4. A similar study on taillight heights was not found in the literature.

TABLE E-4. Sample Headlight Heights for Various Vehicle Types. (*30*)

Vehicle Type	15th Percentile Headlight Height	50th Percentile Headlight Height
Passenger Cars	23.2 in (590 mm)	24.0 in (610 mm)
Light Goods Vehicles	28.0 in (710 mm)	29.1 in (740 mm)
Heavy Goods Vehicles	27.6 in (700 mm)	29.9 in (760 mm)
Articulated Vehicles	28.3 in (720 mm)	30.5 in (775 mm)

Design Criteria Used by Other Countries

For comparison purposes, driver eye and object height criteria used by other countries are presented in Table E-5. Note that most European countries use lower driver eye heights than AASHTO, and that Canada, France, and Great Britain use taillights as their object heights. There also are differences in the taillight height values used by France, Great Britain, and the U.S.

TABLE E-5. Comparison with Other Countries.

Country	Eye Height	Object Height	Headlight Height	Taillight Height	Vehicle Height
Australia	1150 mm	200 mm			
Canada	1050 mm	380 mm		380 mm	
France	1000 mm	350 mm		350 mm	
Germany	1000 mm	150 mm			
Great Britain	1050 mm	260 mm		260 mm	
Sweden	1100 mm	200 mm	600 mm		1350 mm
Switzerland	1000 mm	150 mm			
United States	1070 mm	150 mm	610 mm	457 mm	1300 mm

METHODOLOGY

The methodology described in the following sections documents the data collection, data reduction, and statistical analysis procedures that were followed in this study. Driver eye, headlight, taillight, and vehicle height data were collected under real-world conditions to obtain measured values of driver eye heights when drivers are situated in their vehicle as they would be during normal travel. Data were separated into new vehicles (less than two years old) and older vehicles (more than two years old), in an effort to determine whether the newer vehicle fleet is still demonstrating a downward trend for driver eye height and vehicle height. In addition to the data collection, reduction, and analysis methodology, each of the data collection sites is described in this section.

Data Collection and Reduction Procedures

Three different data collection schemes were developed and used for collecting dynamic vehicle data, heavy vehicle data, and new vehicle fleet data. Initially, the dynamic vehicle study was to provide the driver eye height data for all vehicle types in the current fleet; however, it was determined that driver eye and vehicle height data for passenger cars and heavy vehicles could not be obtained from the same video due to their large differences in heights. The camera had to be level and mounted at about the same height as the height of the object being measured for accurate measurements. Thus, passenger car or heavy vehicle data could be obtained from a single video, but not both.

As a result of this limitation and because passenger cars were of most importance, a second data collection scheme was developed and used to obtain driver eye and vehicle heights for heavy vehicles. This process involved manual measurements of driver eye and vehicle heights of large trucks. The third data collection process involved measuring headlight and taillight heights for every make of vehicle, both foreign and domestic, sold in the United States during the model year 1993.

Dynamic Vehicle Data. The general procedure for collecting the dynamic vehicle data involved videotaping traffic along a roadway such that the vehicle's driver, headlights, taillights, and rooftop could be seen. This process required a two-camera set up, with driver eye, headlight, and vehicle height being obtained from one camera and taillight height being obtained from the second camera. Color cameras were used to maximize contrast as much as possible. Temporary pavement markings were placed on the roadway surface to be used as a scale for data reduction. In addition, reference vehicles were driven through the road segment being filmed to allow for calibration during data reduction. The driver eye height and relevant vehicle dimensions of the reference vehicles were manually measured in the field.

Site Selection. Site selection was one of the critical tasks required for the data collection effort. The characteristics considered in selecting potential study sites are listed below.

- A relatively steady flow of vehicles with a moderate volume was most desirable. Areas with low volume would not generate enough vehicles in a reasonable amount of time, and areas with a higher volume would make data reduction difficult as more than one vehicle over the reference markers at the same time resulted in data that could not be reduced from the video.
- Four-lane (divided or undivided) roadways were desirable to allow data collectors to direct vehicles into the adjacent lane while temporary pavement markings were placed on the roadway surface.
- Steep grades were undesirable because of possible measurement error. A level site was selected in every case.
- Well-lighted roadways were needed for adequate video recording. A cloudy or overcast day would not provide sufficient contrast for driver eye height data collection. The most desirable condition for

data collection was when the sun was positioned such that it shone in the driver's side window.

- The site had to have a good mix of vehicles. An example of a poor site would be a rural highway with a large numbers of pickup trucks and tractor trailers and few passenger cars. A better site would be a suburban type roadway. The surrounding environment was also an important consideration.

Data Collection. The first task in the "set up" procedure for this study was placing the first video camera facing oncoming traffic and as close as practicable to the pavement edge. This camera was used to collect driver eye, headlight, and vehicle height data. Once the first camera position had been established, three parallel strips of temporary pavement marking (reference markers) were placed on the roadway surface, perpendicular to the camera's line of sight and approximately 40 feet (12.2 m) from the camera. The distance between the three reference markers varied from site to site to minimize the spacing and yet still be able to view them as three distinct lines through the video camera. This distance was either 4 or 5 feet (1.2 or 1.5 m) depending on camera placement.

Taillight height data were collected using a second camera. The same basic procedure was followed for set up of the second camera, except only two reference markers were necessary, and the second camera was placed facing departing traffic to record the rear of vehicles. A plan view of a typical camera and reference marker placement is illustrated in Figure E-2, while a photograph of an actual setup is shown in Figure E-3.

Once the cameras had been placed in the proper position for data collection, they were mounted approximately 3.5 feet (1067 mm) above the roadway surface and leveled. The telephoto lenses on each of the cameras were set to manual focuses and adjusted so that vehicles were in focus when they passed over the reference markers. The camera's field of view was set such that both ends of all reference markers were visible and the entire vehicle was visible when crossing over the reference markers. The video camera's shutter speed was set at 1/4000 second to minimize blurring caused by the speed of the vehicles on the video. The high shutter speed also resulted in a shorter travel distance by each vehicle per film frame which was advantageous for data reduction.

After the cameras and reference markers had been placed at the site and the camera adjustments had been made, the site and set up were photographed and a site diagram was prepared for documentation and data reduction purposes. Each site diagram included the distance between each camera and the closest reference marker (measured in the field), and the length and distance between each reference marker. Several additional reference points had to be recorded on the video for data reduction purposes. This procedure involved the use of a tape measure, a surveying level, and a rod with clearly visible marks at 1 feet (300 mm) intervals. With the cameras running, the rod was placed at both ends, the center, and at 6 feet (1.83 m) increments on each of the reference lines. This process added a vertical scale to the horizontal scale provided by the reference markers.

Also with the camera running, all possible combinations of available drivers and vehicles (data collectors and rental vehicles) drove through the site and then had their eye, taillight, headlight, and vehicle heights measured manually and recorded by another person. For example, if four people and two cars were available, eight combinations of drivers and vehicles with known heights were recorded on the video. These "drive throughs" or reference vehicles were used for calibrations during data reduction. After the vertical reference points and the reference vehicles had been recorded, between two and four hours of traffic data were recorded at each site. Consistent with set ups explained previously, driver eye heights, headlights, and tops of free flowing vehicles were recorded on one video camera and taillights were recorded on the second video camera.

Data Reduction. A color monitor and a VCR of frame-by-frame video viewing were used to reduce the data. The data reduction process for each site began by taping a clear plastic sheet on the monitor and then making a template of each site's reference markers. The templates were prepared by carefully placing thin strips of black tape on the plastic sheet on top of the reference markers. Thus, the templates for the driver eye height video tapes contained three black lines and the templates for the taillight height video tapes contained two black lines.

Data reduction began by watching the video tape until a vehicle appeared and neared the reference markers. At this point, the video was stopped using the VCR's frame-by-frame capabilities. This capability allowed the video to be advanced frame-by-frame and was used to place the object of interest (for example, the driver's eye) directly over one of the reference markers. The driver's eye height was measured by placing a metric scale on the video screen at the driver's eye and measuring down to the appropriate reference line. Placing the driver's eye (or any other object such as a taillight, headlights, etc.) directly over a reference line means that the object's height is being measured in the same plane and scale as the reference marker whose measurements are known.

One of the difficulties with this procedure was the ability to place the object of interest directly over one of the reference lines. The three reference lines and the known distances between them were used to aid in positioning the vehicle over a reference marker. An additional aid in positioning the vehicle was the relatively fast shutter speed used to record the video tapes which resulted in at least three frames (positions) between reference markers for most vehicles; however, it should be noted that the object of interest was positioned over the reference markers as closely as could be determined.

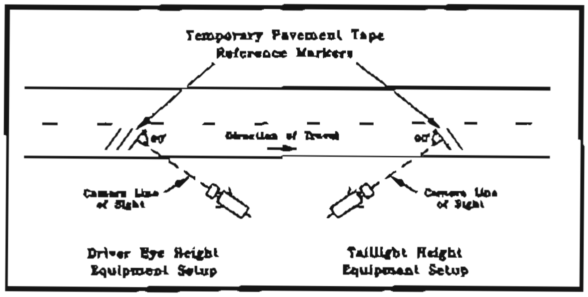


FIGURE E-2. Typical Temporary Pavement Marker and Camera Placement.

FIGURE E-3. Typical Camera Placement.

Of the four items being measured, vehicle height was the most difficult because of the curvature of a vehicle's roof and its appearance on a two-dimensional screen. Generally, the center of the roof is the highest point of the vehicle and the point most desirable to measure; however, this point was difficult to distinguish on video and consequently difficult to position over a reference line. The right edge of each vehicle's roof was a consistently visible point that could be positioned over a reference line. As a result, it was selected as the point for vehicle height measurements. Although the edge of the roof is not the highest point on the vehicle, it should be no more than 1 to 2 inches (25 to 50 mm) lower than the highest point, and this difference could be accounted for with an adjustment to the measured value.

An attempt was made to obtain data from every vehicle that traveled over the reference lines; however, for a variety of reasons, it was not possible to obtain a driver eye, headlight, taillight, and vehicle height data for each and every vehicle. In a number of instances, the vehicle had tinted windows or "flip-up" headlights, or shade or glare made the item to be measured undiscernible on the video tape. Pilot studies indicated that on average, every other vehicle would result in usable data. Thus, twice as many vehicles as needed were videotaped. The inability to measure all items on all vehicles should not introduce any biases in the data set with the possible exception that headlight height data for vehicles with "flip-up" headlights were not included in the database.

In addition to recording driver eye, headlight, taillight, and vehicle height measurements, vehicles were identified by age (less than or more than two years old) and type (passenger car, pick-up truck, minivan, van, sport utility, recreational vehicle, single unit truck, tractor trailer, motorcycle, or bus). Tractor trailers were further categorized as either a "cab-over" or "conventional cab" with respect to their engine. Reference vehicles were also identified by a special code in the database; however, they were not used as data points in the subsequent analysis.

Scaling and Adjustment Factors. The next step in the data reduction process was to determine scaling factors to translate the measured values from the video monitor into their corresponding values in the field. The scaling factors were determined by measuring the length of each reference marker on the video monitor and the length between the 1 foot (300 mm) increments on the surveying rod. The screen measured values for both the reference markers (horizontal reference) and the surveying rods (vertical reference) were related to their actual lengths by dividing the measured value in the field by the measured value from the monitor. Horizontal and vertical scaling factors were averaged if there was a discrepancy between the two frames of reference. Because the reference markers were located at different distances from the cameras, scaling factors were determined for each of the reference markers.

The final step in the data reduction process was to use the reference vehicles at the beginning of the video tapes to determine any necessary corrections to measurements after the scaling factor had been applied. The adjustment factor would be the difference between the two values divided by the field measured value. For example, if a reference vehicle height measured in the field was 1520 mm and the scaled value obtained from the monitor for the same vehicle was 1490 mm, the adjustment factor would be 2.0 percent. Height adjustments were determined for each reference vehicle that was driven through the site and then averaged for each measurement category (headlight height, taillight height, etc.). The average adjustment factor was then applied to all scaled measurements made at the site. Adjustment factors varied from site to site and by measurement category at each of the sites. With one exception, all of the adjustment factors were negative which implied that the scaled heights were greater than the measured heights in the field.

Database. Two databases were created for each site. One database contained the scaled values for driver eye, top of headlight, base of headlight, and vehicle heights for each vehicle. The second database contained the scaled values for the top and base of taillight heights. In addition to the scaled measurements, the database included vehicle type, vehicle age (less than or more than two years old), time that the vehicle appeared on the monitor, and whether the vehicle was a reference vehicle. Reference vehicles were moved to the top of the database and only used to determine the adjustment factors for the scaling values.

The scaled values in the database were multiplied by a scaling factor and the adjustment factor. Thus, each item of interest had three corresponding values in the database, the measured value obtained from the monitor, the scaled value (the measured value with the scaling factor applied), and the adjusted value (the scaled value with the adjustment factor applied). All numeric values in the database were recorded in metric units. The analysis and discussion in the remainder of this appendix refer to adjusted values of driver eye, headlight center, taillight center, and vehicle heights.

Heavy Vehicle Data. The heavy vehicle data collection effort involved manually measuring the driver eye, headlight, taillight, and vehicle height of stationary vehicles at a truck weigh station. The basic procedure involved two data recorders, a surveying rod, and a step ladder. A more detailed description of this procedure is provided in the following paragraphs.

Site Selection. As mentioned, the data collection procedure for the dynamic vehicle data set was not a satisfactory method for collecting driver eye and vehicle height values of heavy vehicles; therefore, it was necessary to use a different procedure to collect driver eye height data for large trucks. The first step in this process was to identify site characteristics where a large number of heavy vehicles and an assortment of truck types would be expected. A truck weigh station that required all heavy trucks on the roadway to enter its facilities seemed like the ideal site. An additional advantage of a truck weigh station is that only heavy vehicles use the facility, minimizing flow disruptions to other vehicles.

Study Procedure. Because it was desirable to collect the heavy vehicle data at truck weigh stations, it was essential that the data collection methodology be efficient to minimize any delay to the truck drivers. Thus, it was decided to use a two person team to manually measure the driver's eye, headlight, taillight, and truck heights. Because of their extreme heights, a surveying rod was used to measure the driver's eye and the truck height. The surveying rod was 13.5 feet (4.11 m) in length when extended and was marked in one tenth of an inch increments. A step ladder was used to place the data collector on the same level as the driver's eye and truck height. Headlight and taillight heights were measured using a meter stick. All values were recorded on data sheets.

Data Reduction. All large truck height values obtained at the weigh station were entered into a database similar to the one used for the dynamic vehicle data. Driver eye and truck height data were entered in the database in English units and then converted to metric units. Headlight and taillight data were entered in the database in metric units only. One difference between the heavy vehicle database and the dynamic vehicle database is the absence of the scaling and adjustment factors in the heavy vehicle data base. Adjustments were not necessary because the measurements in the database were the actual values measured in the field.

1993 New Vehicle Fleet Data. The third data collection effort involved manually measuring the taillight and headlight values of the 1993 vehicle fleet at local dealerships. The basic procedure involved one data collector and a meter stick, and is described in the following sections.

Sample Selection. A cumulative distribution of the 1993 vehicle fleet's headlight and taillight heights was needed to determine if the 1993 vehicle fleet differs from the current vehicle mix. Sales volumes for each domestic and import passenger car or light truck in the 1993 vehicle fleets were obtained from *Automotive News* (*31*). Release of the 1993 vehicle fleet began in August of 1992; therefore, sale volumes were acquired for each month from August 1992 to July 1993. These sale volumes could then be applied as weighting factors to obtain a cumulative distribution of headlight and taillight heights.

Study Procedure. To obtain headlight and taillight measurements of the 1993 vehicle fleet, local automotive dealerships were visited in the Bryan/College Station and Dallas areas. The procedure to obtain the heights of the headlight and taillight involved the use of a meter stick. The height of the top, base, center, and bulb of the headlight and taillight for each vehicle type were measured using the meter stick, and the values were recorded on data sheets.

Data Reduction. The taillight and headlight height values obtained for the 1993 vehicle fleet were entered into a database similar in format to the one used for the dynamic and heavy vehicle data sets. As with the heavy vehicle database, the 1993 vehicle fleet database did not require scaling or adjustment factors because the values in the database were the actual measured values of headlight and taillight height. Cumulative distributions for the 1993 vehicle fleet headlight and taillight heights were determined by matching the measured taillight and headlight height values to individual vehicles in a 2,000 vehicle database in proportion to their percentage of 1993 sales of different vehicle types. Headlight and taillight top, base, center, and bulb heights for 238 vehicle types were entered into the database. It should be noted that there were only 11 vehicle types, representing 0.27 percent of the 1993 vehicles sold, that were not located and, therefore, are not included in the database.

Field Study Sites

In the interest of obtaining a database representative of national conditions, it was decided to obtain driver eye height data from several different geographic regions of the United States. In this way, data from different parts of the country could be compared and it could be determined if the driver eye heights are similar or different from each other. For these reasons, data were collected at sites in four states: Washington, Illinois, Texas, and Virginia. These states represent the northwest, midwest, southwest, and east coast regions, respectively. Site locations are illustrated in Figure E-4 and described in the following sections.

Washington. Although pilot data were collected in College Station, Washington was the first state in which driver eye, headlight, taillight, and vehicle height data were

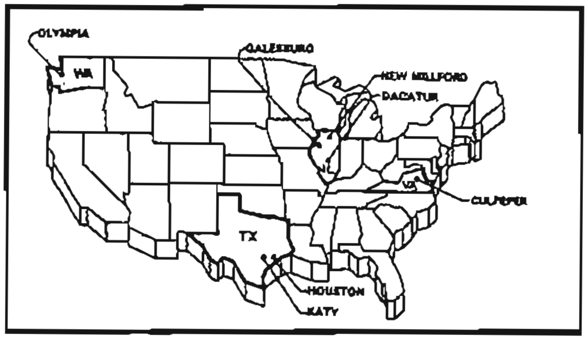


FIGURE E-4. Driver Eye Height Field Study Site Locations.

collected and utilized in this study. The first site chosen for data collection, Black Lake Boulevard, was in the state's capital of Olympia. Black Lake Boulevard is a four-lane divided highway with moderate to high traffic volumes (300 vphpl) and serves as a connector between State Highway 101 and the central business district. The location where data were collected was approximately midway between the state highway and the downtown area. Data collection took place on Saturday, July 31, 1993, and a total of two hours of traffic was taped for analysis. Unfortunately, traffic congestion created problems for data reduction, as queues sometimes extended over the reference markers, hiding them from view, and making it impossible to position vehicles for accurate height measurements during these time periods.

Other difficulties experienced at the Olympia site made data reduction difficult, but not impossible. The video camera was mistakenly left on automatic rather than placed on manual focus, resulting in the video camera constantly refocusing on different vehicles. As a result, the picture's focus constantly changed which means that different scaling factors are required for each vehicle. At first, it was thought that this data could not be reduced from the tape; however, after some thought it was decided to use the vehicle's license plate as the reference to determine a scaling factor and data were recovered for 150 of the 600 vehicles that were on the video tape. The scaled headlight heights for 1993 vehicles at the Olympia site were compared to the measured headlight heights for the same vehicles and found to be in close agreement.

Illinois. The second state where data were collected was Illinois. Three sites were chosen within Illinois, at which two hours of traffic were taped at each. Data collection procedures were revised due to the difficulties experienced in Washington, making the Illinois data collection effort run much smoother.

Decatur. The first data collection site in Illinois was on MacArthur Road in the central Illinois town of Decatur. MacArthur Road is a four-lane undivided road with moderate traffic volumes (175 vphpl) and a posted speed limit of 35 mph. The surrounding area is mostly residential, with an elementary school in the area. The data were collected on Saturday, August 21, 1993, with approximately 350 vehicles captured on video tape. No major difficulties were experienced with data collection or data reduction at this site.

New Millford. The second data collection site in Illinois was on Illinois State Highway 251 in the north central part of the state near the town of New Millford. Although the site was in a mostly rural area, a representative mix of passenger cars and farm-type vehicles was observed. Near New Milford, the state highway is a four-lane undivided roadway with moderate traffic volumes (175 vphpl) and a posted speed limit of 55 mph, making this location the first high speed data collection site. The data were collected at a location just outside the town limits on Tuesday, August 24, 1993, with approximately 350 vehicles captured on video tape. Again, no difficulties were experienced with data collection or data reduction at this site.

Galesburg. The last data collection site in Illinois was on U.S. Highway 150 in the northwestern part of the state near the town of Galesburg. Again, the site was in a mostly rural area, but as at the New Milford site, a representative mix of vehicles was observed. Near Galesburg, U.S. 150 transitions from a two-lane undivided roadway to a four-lane divided highway with low traffic volumes (120 vphpl) and a posted speed limit of 55 mph. Data were collected on the four-lane section of U.S. 150 on Friday, August 27, 1993, with approximately 240 vehicles captured on video tape. One problem with data reduction at this site resulted when the camera filming taillight height data were accidently bumped midway through the data collection, making the last portion of the taillight height data impossible to reduce.

Texas. Two sites were selected for data collection in Texas. The first site's data was used for collecting dynamic vehicle data, while data at the second site became the primary source for the heavy vehicle data. The sites were located in Houston and Katy, respectively.

Houston. The Houston site, McCarty Road, was located near the ship channel in an industrial area of the city. Industries in the area produce a substantial amount of heavy truck traffic; however, there was also a substantial amount of passenger car and light truck traffic in the area. McCarty Road crosses Interstate 10 and provides access to several industrial-type businesses in the ship channel area. The site selected for data collection was a four-lane, undivided road with a posted speed limit of 35 mph. Approximately 3.5 hours of data were collected on Friday, October 1, 1993, with approximately 900 vehicles captured on tape. Traffic volumes were moderate to high, but the high traffic volume levels in this location did not cause problems in data reduction.

A problem with data reduction at the Houston site was caused by the wind turbulence and vibration generated by high speed trucks traveling by the video cameras. The turbulence caused the cameras to move; and, over time, this turbulence changed the camera's field. This problem required the determination of different adjustment factors for different portions of the data. To check the accuracy of the resultant adjustment factors, a sample of fifteen 1993 vehicles was identified on the video tape and the scaled headlight (or taillight) heights of the vehicles were compared with the heights measured at an auto dealer. The average difference between the heights measured from the video tape and the heights measured at the auto dealers were used as the height adjustment factors. The height adjustments for driver eye height, headlight height, and vehicle height differed between the two methods by less than 4 percent.

Katy. The second data collection site in Texas was a truck weigh station located in a rural area approximately 40 miles west of Houston and adjacent to Interstate 10. Interstate 10 is the major truck route between Houston and San Antonio. As a result, it was expected that this weigh station would produce a large number and variety of truck traffic. A sign with flashers on Interstate 10 indicating that trucks were required to use the weigh station facilities was activated during time that data were collected. There were times when data collectors could not measure and record data at that same rate as trucks were entering the weigh station and the flashers on Interstate 10 were turned off. Data were collected for approximately 4.5 hours on Friday, February 25, 1994, resulting in data for 164 trucks being including in the final data set from this site.

Virginia. The last site at which video data were collected was located in north central Virginia near the town of Culpeper. The site selected for data collection was located on a suburban section of State Highway 229, a four-lane divided roadway with a posted speed limit of 45 mph. Unfortunately, it rained on the day data were collected, making data collection and reduction difficult because of poor visibility. In anticipation of problem with data reduction, four hours of traffic data were collected. More data would have been collected; however, the weather worsened and data collection was halted. The visibility problems caused by the weather conditions resulted in driver eye heights for only 150 of the approximately 740 vehicles on the Culpeper video tape being included in the final database.

RESULTS

The driver eye and vehicle height results are presented by vehicle type followed by comparisons between geographic regions and age of vehicles. Although several different vehicle types were observed during data reduction, the data were aggregated into three major groups, namely the passenger car, multipurpose vehicle, and tractor trailer. Multipurpose vehicles consist of pick-up trucks, sport utility vehicles, minivans, and vans. Other types of vehicles observed during data collection were single-unit commercial vehicles, buses, and motorcycles.

Figure E-5a illustrates the distribution of vehicle types represented in the dynamic vehicle data set of 2,696 vehicles. Of the vehicles in the database, approximately 50 percent were passenger cars and 37 percent were multipurpose vehicles; however, as mentioned in the *Data Reduction Procedure* section, there is not a driver eye, headlight, taillight, and vehicle height associated with every vehicle as it was not possible to measure all four heights for each vehicle.

The 1993 vehicle fleet data set represents the distribution of passenger cars and multipurpose vehicles sold in 1993 (see Figure E-5b). As defined above, the multipurpose category includes pick-up trucks, minivans, vans, and sport utilities. Note that 61.7 percent of the new vehicles were passenger cars and 38.3 percent of the new vehicles were multipurpose vehicles, and also that these data did not include any of the large commercial trucks sold in 1993. This distribution is similar to the proportion of passenger cars and multipurpose vehicles in the dynamic vehicle data set.

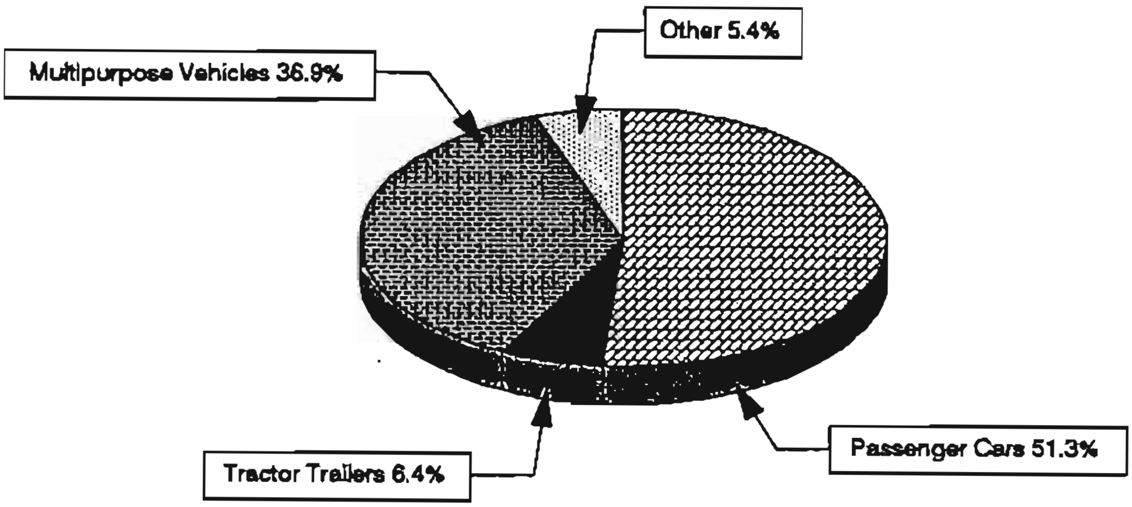
A distribution of automobiles and trucks currently operating in the U.S. was obtained from a representative of the *Ward's Automotive Yearbook*. The truck category represents all trucks having a gross vehicle weight between 1 and 8 tons (900 and 7260 kg). Thus, this category includes vehicles from full-size pick-up trucks to large tractor units; however, it does not contain the majority of the multipurpose vehicles that were observed in this study. Those vehicles would be included in the passenger car category. Figure E-5c illustrates the distribution of vehicle types currently operating in the U.S. Based on this data, it appears that passenger cars represent approximately half to two-thirds of the vehicle fleet.

Passenger Cars

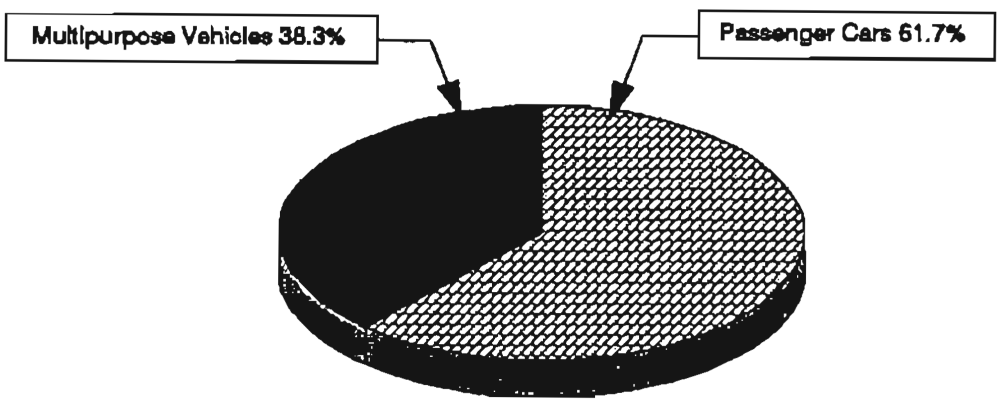
Table E-6 presents summary statistics for the passenger cars included in the dynamic vehicle data set. The driver eye, headlight, and vehicle height results are based on data from four states (WA, IL, TX, and VA), while the taillight results are based on data from three states (IL, TX, and VA). Headlight and taillight heights represent the center of the light as specified by *Federal Motor Vehicle Standard 108*.

Of the 875 passenger cars in the driver eye height database, the 5th percentile value of 3.48 feet (1,060 mm) was very close to the current driver eye height of 3.5 feet (1,067 mm). Passenger car driver eye height values ranged from a high of (1,422 mm) to a low of (955 mm) with 50th and 15th percentile values of (1,149 mm) and (1094 mm), respectively. Percentile values refer to the percentage of the total observations that were below these values.

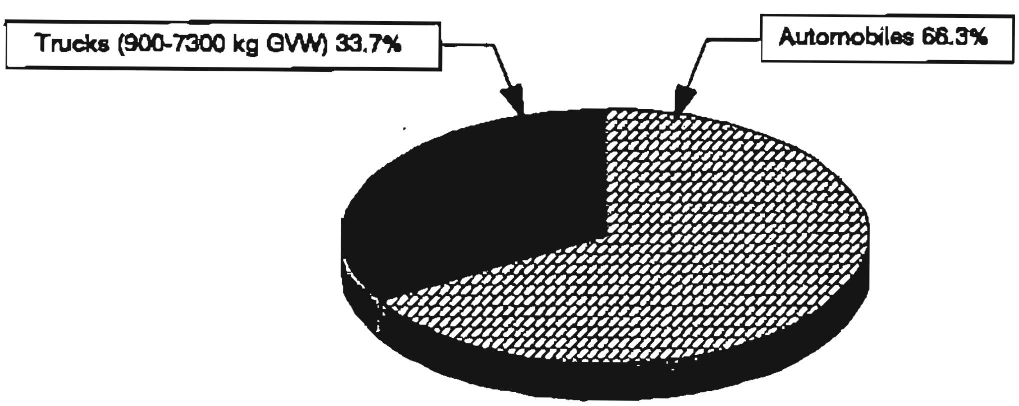
Of the 1,318 passenger cars in the headlight height database, less than 1 percent of them were below the 22 inches (560 mm) minimum height requirement of *Federal Motor Vehicle Standard 108*; however, these few vehicles were all within 0.79 inches (20 mm) of the standard. None of the headlights measured were above the 54 inches (1,372 mm) requirement. Headlight height values ranged from a high of (947 mm) to a low of (541 mm) with 50th, 15th, and 5th percentile values of (649 mm), (608 mm), and (590 mm), respectively.



(a) Dynamic Vehicle Fleet.



(b) 1993 Vehicle Fleet.



(c) Current Vehicle Fleet.

FIGURE E-5. Distribution of Vehicle Types.

TABLE E-6. Descriptive Statistics for Passenger Cars.

Descriptive Statistic*	Driver Eye Height	Headlight Height	Taillight Height	Vehicle Height
Sample Size	875	1318	858	1378
Mean	1149 (3.77)	649 (2.13)	726 (2.38)	1384 (4.54)
Standard Deviation	55 (0.18)	41 (0.13)	70 (0.23)	59 (0.19)
High Value	1422 (4.67)	947 (3.11)	999 (3.28)	1690 (5.54)
Low Value	955 (3.13)	541 (1.77)	385 (1.26)	1156 (3.79)
Range	467 (1.53)	406 (1.33)	614 (2.01)	534 (1.75)
5th Percentile	1060 (3.48)	590 (1.94)	616 (2.02)	1282 (4.21)
10th Percentile	1082 (3.55)	602 (1.98)	642 (2.11)	1315 (4.31)
15th Percentile	1094 (3.59)	608 (1.99)	660 (2.17)	1331 (4.37)

* *Descriptive Statistics presented in millimeters (and feet) where applicable.*

Of the 858 passenger cars in the taillight height database, none of them were lower than the 15 inch (80 mm) requirement or higher than the 72 inch (1830 mm) requirement in *Federal Motor Vehicle Standard 108*. Taillight height values ranged from a high of (999 mm) to a low of (385 mm) with 50th, 15th, and 5th percentile values of (726 mm), (660 mm), and (616 mm), respectively.

The 1,378 passenger car heights in the vehicle height database ranged from a high of (1,690 mm) to a low of (1,156 mm) with 50th, 15th, and 5th percentile values of (1,384 mm), (1,331 mm), and (1,282 mm), respectively. A height of 4.25 feet (1,300 mm) is used to establish design criteria for passing and intersection sight distances, and more than 90 percent of the 1,378 vehicles in this study's database had heights greater than 1,300 mm.

Multipurpose Vehicles

Table E-7, on the previous page, presents the summary statistics for the multipurpose vehicles in the dynamic vehicle data set. As mentioned, the multipurpose vehicle category contained pick-up trucks, sport utility vehicles, minivans, and vans, and represented almost 37 percent of the observations in this study's database. The 5th percentile driver eye height for multipurpose vehicles was 4.15 feet (1264 mm), which was 2 feet (204 mm) greater than the 5th percentile driver eye height for passenger cars. Also, the lowest driver eye height for multipurpose vehicles was approximately equal to the 5th percentile driver eye height for passenger cars.

The lowest values measured for multipurpose vehicle headlight and vehicle heights were also approximately equal to the 5th percentile headlight and vehicle heights for passenger cars. The 5th percentile headlight and vehicle heights for multipurpose vehicles were 27.2 inches (691 mm) and 5 feet (1523 mm), respectively. The taillight heights for the vehicles in the multipurpose vehicle category were also higher than the passenger car values. The 5th percentile taillight height was (780 mm), which was (164 mm) above the 5th percentile passenger car value

Heavy Trucks

Table E-8 presents summary statistics for the heavy trucks included in this study's database. Note that this vehicle category only includes tractor-trailer combination vehicles. The 50th, 15th, and 5th percentile eye heights for the 163 trucks in the database were 8.0 feet (2447 mm), (2341 mm) and (2304 mm), respectively. The lowest tractor-trailer eye height was higher than the highest value for a passenger car. The headlight and taillight height values for tractor-trailers were all within the *Federal Motor Vehicle Standard 108* requirements. The highest point on the tractor or trailer was used in the vehicle height measurement. The 5th percentile vehicle height was (2652 mm) and the 85th percentile height was 13.3 feet (4054 mm), both slightly lower than the maximum legal vehicle height in Texas 13.5 feet.

In addition to the truck height measurements, the driver eye height of "cab-over" versus "conventional cab" tractor trailers was investigated. Of the 159 tractor trailers that could be categorized as either cab-over or conventional cab tractors, 130 represented the conventional cab tractor trailers. The mean driver eye height was similar for the two types of truck; however, the 15th percentile driver eye height was 2 percent lower for conventional cab trucks than for cab-over trucks. Descriptive statistics for the two truck types are presented in Table E-9.

TABLE E-7. Descriptive Statistics for Multipurpose Vehicles.

Descriptive Statistic*	Driver Eye Height	Headlight Height	Taillight Height	Vehicle Height
Sample Size	629	992	534	987
Mean	1482 (4.86)	842 (2.76)	963 (3.16)	1759 (5.77)
Standard Deviation	130 (0.43)	95 (0.31)	132 (0.43)	155 (0.51)
High Value	2034 (6.67)	1174 (3.85)	1436 (4.71)	2501 (8.21)
Low Value	1053 (3.45)	569 (1.87)	420 (1.38)	1279 (4.20)
Range	981 (3.22)	605 (1.98)	1016 (3.33)	1222 (4.01)
5th Percentile	1264 (4.15)	691 (2.27)	780 (2.56)	1523 (5.00)
10th Percentile	1306 (4.28)	713 (2.34)	818 (2.68)	1564 (5.13)
15th Percentile	1331 (4.37)	728 (2.39)	839 (2.75)	1613 (5.29)

* *Descriptive Statistics presented in millimeters (and feet) where applicable.*

TABLE E-8. Descriptive Statistics for Heavy Trucks.

Descriptive Statistic*	Driver Eye Height	Headlight Height	Taillight Height	Vehicle Height
Sample Size	163	337	260	158
Mean	2447 (8.03)	1121 (3.68)	1058 (3.47)	3590 (11.78)
Standard Deviation	107 (0.35)	88 (0.29)	159 (0.52)	581 (1.91)
High Value	2816 (9.24)	1351 (4.43)	1690 (5.54)	4639 (15.22)
Low Value	2103 (6.90)	915 (3.00)	415 (1.36)	2396 (7.86)
Range	713 (2.34)	436 (1.43)	1275 (4.18)	2243 (7.36)
5th Percentile	2304 (7.56)	972 (3.19)	719 (2.36)	2652 (8.71)
10th Percentile	2329 (7.64)	1008 (3.31)	908 (2.98)	2719 (8.92)
15th Percentile	2341 (7.68)	1022 (3.35)	953 (3.13)	2774 (9.10)
85th Percentile	2560 (8.40)	1220 (4.00)	1185 (3.89)	4054 (13.30)
90th Percentile	2579 (8.46)	1236 (4.06)	1208 (3.96)	4084 (13.40)
95th Percentile	2597 (8.52)	1258 (4.13)	1265 (4.15)	4084 (13.40)

* *Descriptive Statistics presented in millimeters (and feet) where applicable.*

TABLE E-9. Driver Eye Height Descriptive Statistics for Heavy Truck Types.

Descriptive Statistic*	Cab-Over	Conventional Cab
Sample Size	29	130
Mean	2443 (8.02)	2445 (8.02)
Standard Deviation	95 (0.31)	109 (0.36)
High Value	2652 (8.70)	2816 (9.24)
Low Value	2134 (7.00)	2103 (6.90)
Range	518 (1.70)	713 (2.34)
5th Percentile	2341 (7.68)	2304 (7.56)
10th Percentile	2365 (7.76)	2323 (7.62)
15th Percentile	*2384 (7.82)*	*2335 (7.66)*

* *Descriptive Statistics presented in millimeters (and feet) where applicable.*

Comparison by Vehicle Type

Figure E-6 illustrates the driver eye height distribution for passenger cars, multipurpose vehicles, and heavy trucks as well as the current AASHTO assumption for driver eye height. Note that approximately 95 percent of the passenger cars and all of the multipurpose and heavy trucks in the dynamic vehicle data set had driver eye heights greater than the value currently being used to establish geometric design criteria.

Figure E-7 illustrates the headlight height distribution for the three types of vehicles. All of the vehicles in the data base had headlight heights that were less than the maximum requirement. The graph indicates that less than approximately 1 percent of the passenger cars had headlights which were lower than the minimum headlight height requirement of 22 inches (560 mm). The headlight height distributions differ noticeably by vehicle type. The lowest measured heavy truck headlight height was about the same as the highest measured passenger car headlight height.

Figure E-8 illustrates the taillight height distributions for passenger cars, multipurpose vehicles, and heavy trucks. The minimum taillight height requirement is also shown in the figure, but because of scale, the maximum taillight height requirement of 6 feet (1830 mm) is not shown. All vehicle taillights observed in this study were between the minimum and maximum height requirements. In general, multipurpose vehicles have higher taillight heights than passenger cars, and heavy trucks have higher taillight heights than both passenger cars and multipurpose vehicles.

Figure E-9 illustrates the vehicle height distributions for the three types of vehicles in the study's data base. As expected, the height of heavy trucks was much greater than the heights of passenger cars and multipurpose vehicles. Approximately 99 percent (162 out of 163 observations) of the observed heavy truck heights were below the maximum legal height of 13.5 feet (4115 mm).

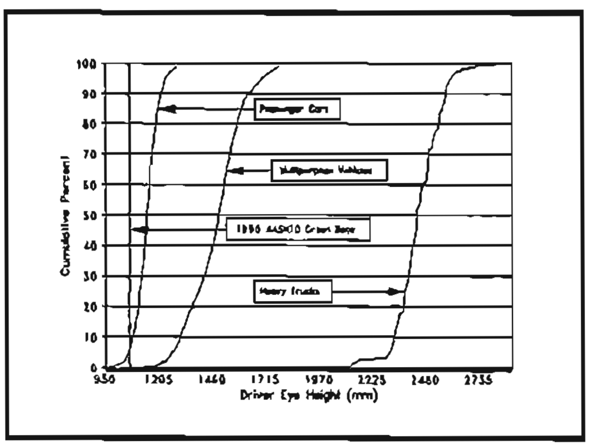


FIGURE E-6. Driver Eye Height Distributions by Vehicle Type.

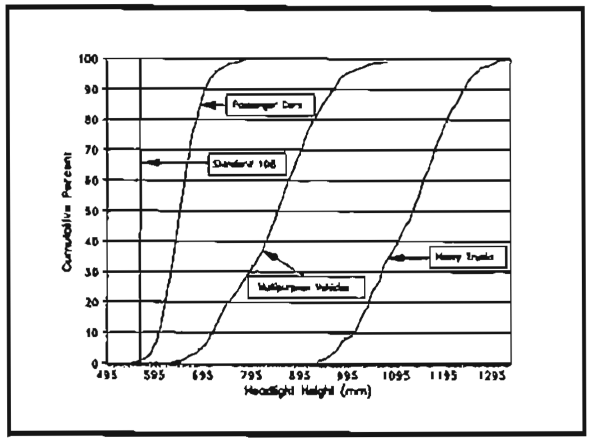


FIGURE E-7. Headlight Height Distributions by Vehicle Type.

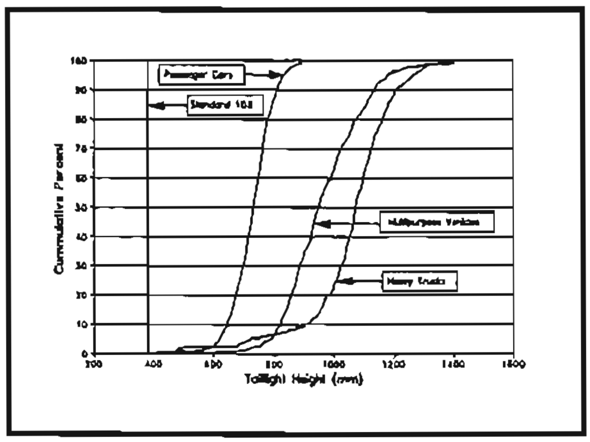


FIGURE E-8. Taillight Height Distributions by Vehicle Type.

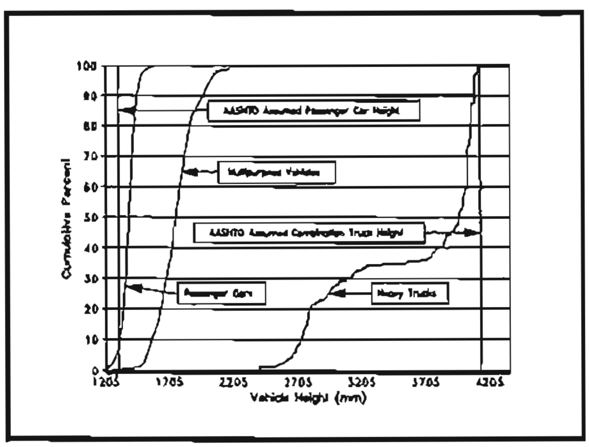


FIGURE E-9. Vehicle Height Distributions by Vehicle Type.

Statistical Comparison

The uses of statistical tests for comparing two population means generally require that the samples being compared have the same probability distribution; therefore, it was necessary to test this assumption before making a statistical comparison. All driver eye, headlight, taillight, and vehicle height distributions were normally distributed with the exception of the 1993 vehicle fleet headlight and taillight data. Thus, standard parametric and non-parametric analysis of variance techniques were used to compare sample means. For this study, statistical comparisons were made between headlight and taillight values by age of the vehicle and between driver eye heights by geographic region (state).

Comparison by Age of Passenger Car. The age of each passenger car in the dynamic vehicle data set was estimated as either less than or more than two years old. This classification allowed a comparison of driver eye, headlight, taillight, and vehicle heights between new and older vehicles. Such comparisons would be useful in determining whether these heights are changing for the newer vehicle fleet. The driver eye height distributions for the two different vehicle age groups are shown in Figure E-10. The figure and a statistical comparison of the means of the two distributions did not suggest a difference in driver eye heights as a result of the age of the vehicle.

The plot of the passenger car headlight heights, however, illustrates a different trend than the driver eye height data (see Figure E-11). Passenger cars less than two years old had lower headlight heights than passenger cars more than two years old. The 1993 passenger car fleet also had lower headlight heights than both age categories in the dynamic passenger car data. This difference was significant at the 95 percent confidence level. Given that the *Federal Motor Vehicle Standard 108* headlight height requirements decreased from 24 inches (610 mm) to 22 inches (560 mm) in 1985, this finding is not surprising. Even though headlight heights are decreasing for newer vehicles, they are not expected to decrease below the minimum height requirements of 22 inches (560 mm) unless the current vehicle standards change.

When taillight heights for the two age groups in the dynamic vehicle data base and the 1993 passenger car data base were plotted (see Figure E-12) a somewhat different trend appeared. The newer vehicles in the dynamic vehicle data base had higher taillight heights than the older vehicles, and the 1993 vehicle fleet had even higher taillight heights. These differences were significant at the 95 percent confidence level. This finding indicates that passenger car taillight heights are increasing for newer vehicles, and this trend continued for the 1993 new vehicle fleet; however, from a practical standpoint, increases in taillight heights will probably not continue indefinitely.

Comparison with 1993 Vehicle Fleet. Headlight and taillight height descriptive statistics for passenger cars, multipurpose vehicles, and a combination of both vehicle types for both the dynamic vehicle data base and the 1993

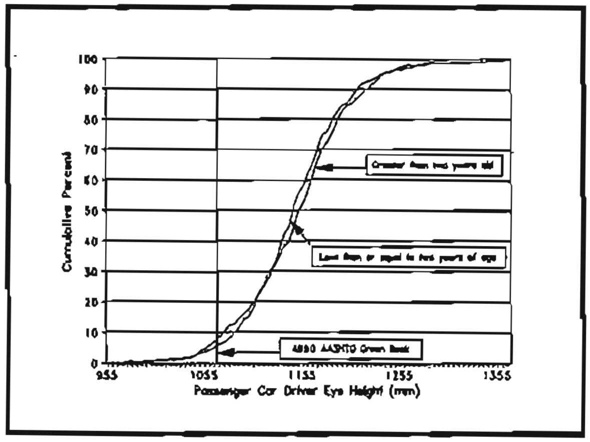


FIGURE E-10. Passenger Car Driver Eye Height Distribution by Vehicle Age.

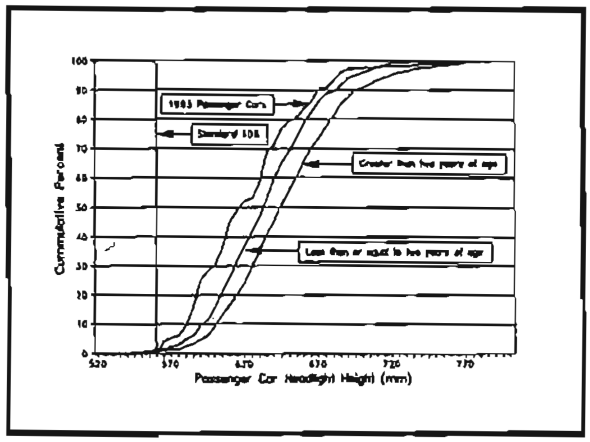


FIGURE E-11. Passenger Car Headlight Height Distributions by Vehicle Age.

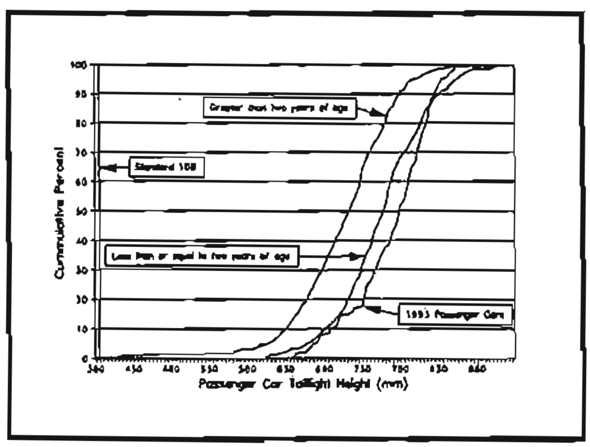


FIGURE E-12. Passenger Car Taillight Distributions by Vehicle Age.

vehicle fleet are presented in Table E-10. As shown, there is very little difference between the 5th percentile headlight height for passenger cars and that of the combination of passenger cars and multipurpose vehicles (approximately 10 mm or 0.4 in) for either the dynamic or 1993 vehicle fleet data.

For comparison purposes, 5th percentile taillight height for passenger cars and the combination of passenger cars and multipurpose vehicles differed by less than 0.8 inches (20 mm) for both the dynamic vehicle data and the 1993 vehicle fleet. Note that the 5th percentile taillight height for 1993 vehicles is approximately 10 percent higher than the dynamic vehicle fleet. This data supports the finding that taillight heights are increasing as vehicle age decreases.

Comparison by State. To determine if there were regional differences in the dynamic vehicle data base, the driver eye and taillight height data from the different states were compared. The analysis of the driver eye height data revealed significant differences in mean driver eye heights—driver eye heights were lower in Texas than they were in Washington, Illinois, and Virginia. The magnitude of this difference was only 1.4 inches (35 mm) and is not large enough to be of concern. The analysis of the taillight data revealed no differences in mean taillight height for the four states.

SUMMARY

The field studies documented in this appendix examined driver eye, headlight, taillight, and vehicle height values for establishing geometric design criteria for highways and streets. The literature was reviewed to document what had been done in the past, and field data were collected to determine current values of these parameters. Findings and recommendations from the driver eye and vehicle height field studies are presented in the following section.

Findings

- Of the 1667 passenger car, multipurpose vehicle, and large truck driver eye heights in the study's data base, more than 97 percent were higher than the 1994 AASHTO design value of 1070 mm. Of the 875 passenger car driver eye heights included in the data base, the 5th and 15th percentile driver eye heights were 1066 mm and 1094 mm, respectively. The 5th and 15th percentile driver eye heights for the 163 large trucks were 2304 mm and 2560 mm, respectively.
- Of the 1318 passenger car headlight heights in the study's data base, 10 were below the 22 inch (560 mm) requirement of *Federal Motor Vehicle Standards 108*. No multipurpose vehicles or large trucks had headlight heights below this standard. The 5th and 15th percentile passenger car headlight heights were 590 mm and 608 mm, respectively.
- A significant decrease in headlight heights was noted between older passenger cars and the 1993 vehicle fleet. It could be concluded that headlight heights are decreasing on newer passenger cars; however, it should be noted that at least part of this decrease can be attributed to changing the *Standard 108* requirement from 24 inch (610 mm) to 560 mm (22 inches) in 1985. It is not anticipated that passenger car headlight heights will continue the downward trend unless *Standard 108* is changed.

TABLE E-10. Headlight and Taillight Height Descriptive Statistics.

		Dynamic Vehicle Fleet			1993 Vehicle Fleet		
		PC	MV	PC & MV	PC	MV	PC & MV
Headlight Statistics*	Mean	649 (2.13)	842 (2.76)	732 (2.40)	634 (2.08)	830 (2.72)	709 (2.33)
	50th Percentile	646 (2.12)	847 (2.78)	688 (2.26)	630 (2.07)	835 (2.74)	660 (2.17)
	15th Percentile	608 (1.99)	728 (2.39)	623 (2.04)	595 (1.95)	760 (2.49)	600 (1.97)
	5th Percentile	590 (1.94)	691 (2.27)	599 (1.97)	580 (1.90)	710 (2.33)	590 (1.94)
Taillight statistics*	Mean	726 (2.38)	963 (3.16)	817 (2.68)	771 (2.53)	957 (3.14)	842 (2.76)
	50th Percentile	728 (2.39)	949 (3.11)	774 (2.54)	780 (2.56)	955 (3.13)	810 (2.66)
	15th Percentile	660 (2.17)	839 (2.75)	682 (2.24)	715 (2.35)	835 (2.74)	735 (2.41)
	5th Percentile	616 (2.02)	780 (2.56)	632 (2.07)	665 (2.18)	798 (2.62)	678 (2.22)

* *Descriptive Statistics presented in millimeters (and feet).*
** *PC = Passenger Cars, MV = Multipurpose Vehicles*

- Of the 1652 taillight heights in the study's data base, none were below the *Standard 108* requirement of 15 inch (380 mm). Of the 858 passenger car taillight heights included in the data base, the 5th and 15th percentile were 616 mm and 660 mm, respectively.

- A significant increase in taillight heights was noted between older passenger cars and the newer passenger cars. It could be concluded that taillight heights are increasing on newer passenger cars; however, this increase is not expected to continue indefinitely.

- Of the 1378 passenger car vehicle heights in the study's data base, the 5th and 15th percentile values were 1282 mm and 1331 mm, respectively. Of the 158 large trucks in the data base, 157 had vehicle heights less than or equal to the 13.5 feet (4115-mm) maximum legal height for these vehicles. The 95th percentile vehicle height value for large trucks determined by this study was 13.4 feet (4084 mm).

Recommendations

When driver eye, headlight, taillight, and vehicle heights are used to establish geometric design criteria, the following recommendations are suggested:

- A driver eye height of either 1080 mm is recommended for design purposes. This value represents 90 percent of the passenger car driver eye height values. It also represents an even higher percentage of the total vehicle fleet because passenger cars, which have the lowest driver eye height values, represent less than two-thirds of the total vehicle fleet.

- A headlight height (measured to the center of the headlight) of 600 mm, the 95th percentile value, (560 mm) or 22 inches, the minimum permissible value of headlight height allowed by *Standard 108*, is recommended. The latter value represents more than 99 percent of the passenger car headlight heights observed in this study.

- A taillight height of 23.8 inches (600 mm) is recommended. Although this value is substantially higher than the *Standard 108* value of 15 inches (380 mm), the data obtained in this study indicate the trend of taillight height is increasing, and no taillight height values for any vehicle type were found to be below the *Standard 108* requirement. The recommended value represents more than 95 percent of the passenger car taillight height values observed in this study.

- A passenger car height value of either 1280 mm of 1330 mm is recommended. These values represent the 5th and 15th percentile passenger car height values, respectively. Because a concern for heavy trucks relates to their maximum rather than their minimum heights, a vehicle height of 13.5 feet (4115 mm) is recommended. This value represents the current maximum legal height value for large trucks.

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APPENDIX F -

SAFETY STUDIES

INTRODUCTION

For safe and efficient transportation systems, stopping sight distance model (SSD model) parameters should be representative of realistic conditions for the current driving environment. For example, if required stopping sight distance values do not reflect driver needs, drivers may not have enough time to identify potential hazards in the roadway and respond by stopping or going around the hazard (i.e., an accident may occur); however, if the required stopping sight distance values are beyond the driver's physical capabilities, available stopping sight distance may not be a contributing factor to the accident. A thorough examination of the accidents on roadways with limited stopping sight distance is necessary to answer whether the SSD model represents realistic conditions in terms of the effects on safety.

The objective of the accident study was to determine if stopping sight distance was a contributing factor in accidents on limited sight distance roadways. In order to accomplish this objective, the following tasks were performed:

- Reviewed previous literature on sight distance and safety on limited sight distance roadway segments.
- Identified roadways that contain limited sight distance crest vertical curves.
- Analyzed accident narratives to determine if sight distance is a contributing factor on roadways with limited sight distance.
- Identified patterns and/or characteristics of roadway segments that contain accidents with limited sight distance as a contributing factor.

The findings from the accident study are divided into five sections. Section 1, *Introduction*, presents the introduction and objectives of the accident studies, and the organization of this appendix. Section 2, *Literature Review*, describes previous research on accident studies and improvements for roadways with limited sight distance crest vertical curves. Section 3, *Methodology*, describes the data collection, accident analysis, and data analysis procedures. Section 4, *Results*, documents the results of the accident study, and the findings and recommendations of the study are described in the Section 5, *Conclusions and Recommendations*.

LITERATURE REVIEW

Common sense suggests that providing less stopping sight distance than needed should cause a safety problem, and that providing more stopping sight distance than needed should not cause a safety problem. The challenge for researchers and practitioners is defining what stopping sight distance value is adequate. Thus, a logical starting point for this study is a review of past research on the relationship between sight distance and safety. The first part of the literature review contains a discussion on the adequacy and accessibility of existing local, state, and national data bases to develop a relationship between sight distance and safety. This discussion is followed by the limitations of using accident data for developing such a relationship.

The remainder of this section provides summaries of previous research on accident studies at limited sight distance crest vertical curves, and related studies on truck accidents, object-related accidents, and operating speeds at limited sight distance crest vertical curves. The final part of this section summarizes some of the important points from the literature review and presents a conceptual model of the relationship between available stopping sight distance and safety at crest vertical curves.

Accident Data Base Requirements

The first question to be answered is, What are the accident data base requirements for a study of this type? Realistically, an accident data base for developing a relationship between stopping sight distance and safety, or the validity of various hypothesized SSD models, would have to contain the following information as a minimum:

- A detailed, fine-grained description of the highway environment (i.e., traffic volumes, available sight distance, roadway geometry, etc.) **by location** (i.e., by milepoint, milepost, coordinates, link-and-node, etc.); and
- A large, detailed set of accident data that can be merged with the highway environmental data **by location**.

For example, imagine a data base containing traffic volumes, available sight distance, roadway geometry, etc., at 0.1-mile intervals for 100 miles of highway. Thus, the data set would contain 1,000 records on highway characteristics. Now, imagine taking five years of accident data and merging it by location to the 1,000 highway locations referred to previously. Theoretically, the resultant merged **location-based accident data file** could then be used to develop relationships between stopping sight distance and safety (through fairly standard multiple regression techniques), or to assess the validity of various hypothesized SSD models (assuming that all variables in these models are contained within the location-based accident data file).

It follows from the two requirements listed above that only statewide accident data bases (that can be merged to traffic and geometric data files) offer the potential to develop relationships between stopping sight distance and safety, or to assess the validity of various hypothesized SSD models. Other **accident-based accident data files** (i.e., the National Accident Sampling System [NASS] and the Fatal Accident Reporting System [FARS]) are not suitable for this type of study. Similarly, the Highway Performance Monitoring System (HPMS), which is a location-based accident data file, is not suitable because it does not contain "fine-grained" information, i.e., individual accidents in this data set cannot be associated with specific roadway features or characteristics at the accident location.

All 50 states have developed automated traffic accident data bases to document motor vehicle accidents within their jurisdictions. Many of the states are able to merge their accident data files with roadway-information files to produce the location-based accident files discussed above. Furthermore, the Federal Highway Administration (FHWA), through a contract with the Highway Safety Research Center of the University of North Carolina, has constructed a Highway Safety Information System (HSIS) containing five years of accident data collected in five different states. HSIS can be used to generate location-based accident data files that meet the two requirements listed previously.

HSIS is a computerized accident data base that includes data from five states: Illinois, Maine, Michigan, Minnesota, and Utah. The primary criteria used in selecting the states were the data availability (the range of data variables collected), quantity, and quality. HSIS allows locations to be identified based on roadway variables, and accident and traffic data to be attached to these locations. Within HSIS, accident, roadway inventory, traffic volume, roadway geometry, intersection detail, and guardrail files are available although not all files are available for each of the states. A similar computerized accident data base, including roadway geometry, exists for Texas.

Accident Data Base Limitations

In theory, the location-based accident data files compiled from state accident records could be used to develop relationships between stopping sight distance and safety or to assess the validity of various hypothesized SSD models; however, there are several limitations associated with accident data that should be addressed. Some of the difficulties in developing these relationships and validating SSD models are discussed in the following sections.

Accident Location. The key to any study of the relationship between stopping sight distance and safety is a definitive knowledge of where the accident occurred. Consider the following facts:

- Police officers are typically instructed to record the accident location in terms of the "first harmful event" in the accident. This information is typically entered into the accident data base to the nearest 0.1 miles (i.e., to the nearest 528 feet).
- Although police officers are instructed to define the accident location in terms of "first harmful event," many officers no doubt record accident location in terms of "final rest." For a car that leaves the highway at 60 miles per hour (88 feet/second), the difference in location between "first harmful event" and "final rest" may be significant. Location of the accident should be identified in reference to the highway from which the vehicle departed, and at the point on the highway that is nearest to the damage or injury producing events.
- Further, a common practice of police officers is diluting the precision of traffic accident location by "estimating" distance; i.e., the accident occurred 0.5 miles west of a particular intersection. When police accident reports are reviewed, it is more common to find that an accident occurred 0.5 miles (or 1.0 miles) from an intersection, rather than 0.4 miles (or 1.1 miles) from an intersection. Such evidence suggests that "accident location" might more appropriately be referred to as "estimated accident location."
- Finally, it should be recognized that the highway location that precipitates an accident (i.e., the approach to a crest vertical curve) and the "first harmful event" in the accident may be several hundred feet apart. Thus, it is quite possible that a given accident occurred at a location where stopping sight distance was adequate, but was initiated at a location where stopping sight distance was inadequate, and vice versa.

Accident Subsets. Traffic accidents are the result of many contributing factors acting singly and in combination with one another. For this reason, when transportation analysts are called upon to assess the safety benefits of different roadway features or devices, the first question they should ask is, What kinds of accidents are affected by the feature or device being evaluated?

For example, imagine being asked to evaluate the safety benefits of a pavement resurfacing project. The first question that should be asked is, What kinds of accidents are affected by resurfacing projects? If this question was not asked and the resurfacing project evaluated in terms of *total* accidents, it probably would have been concluded that pavement resurfacing had little effect on safety. On the other hand, if the project had been evaluated in terms of *wet-surface* accidents, it probably would have been concluded that pavement resurfacing had a positive effect on safety. Thus, when evaluating the safety benefits of resurfacing projects, the number of *total* accidents is an insensitive dependent variable and the number of *wet-surface* accidents is a sensitive dependent variable. It follows that by looking specifically at the subset of accidents that resurfacing projects are intended to address, more leverage is available for assessing the safety benefits.

By analogy, the ability to develop relationships between accident frequency and stopping sight distance would be enhanced if the types of accidents affected by stopping sight distance could be defined. Unfortunately, limited stopping sight distance could be associated with many different types of accidents: accidents involving objects in the roadway, accidents involving animals in the roadway, accidents involving vehicles at intersections, head-on accidents involving improper passing, etc. Compounding the problem, accidents involving objects near crest vertical curves might be affected by available stopping sight distance, whereas accidents involving objects on straight flat sections of roadway are probably not affected by available stopping sight distance. The absence of a clearly defined subset of accidents associated with limited stopping sight distance hinders the ability to develop relationships between available stopping sight distance and safety.

Accident Frequency. Most roadways characterized as having limited stopping sight distance vertical curves are typically older, lower volume highways. Generally, roadways that have been recently constructed or that carry higher volumes of traffic have been built or upgraded to higher design standards. Given that the best predictor of future accidents is traffic volume and that most examples of limited stopping sight distance are found on relatively low-volume highways, it follows that available accident data for assessing deficiencies in stopping sight distance will be relatively sparse. This sparsity of data has profound implications for any analyses that would be carried out to determine the relationship between stopping sight distance and safety. Consider the following simplistic, hypothetical example.

Imagine having two equivalent 50-mile segments of highway on two-lane rural roads, each carrying less than 1,000 vehicles per day. The first segment is characterized by *adequate stopping sight distance*, and the second is characterized by *inadequate stopping sight distance*. Now suppose that it is an indisputable fact that *inadequate stopping sight distance* will produce a 20 percent increase in accidents. The question then becomes, What is the expected number of accidents per year on these two highway segments and is that difference large enough to be statistically significant?

To provide some numbers for illustration, there are approximately 0.2 accidents per mile per year on Texas two-lane rural highways carrying less than 1,000 vehicles per day. Thus, on the first highway segment with adequate stopping sight distance, ten accidents might be expected, and on the second segment with inadequate stopping sight distance (20 percent more accidents), 12 accidents might be expected. This relatively small difference in accident frequency between the two highway segments would not be statistically significant even with the indisputable information of a 20 percent increase in accidents. That is, for small sample sizes, a difference of two accidents per year is well within the limits of chance expectation.

To overcome the problem of small or inadequate sample size, accident-based analyses of the relationship between stopping sight distance and accidents must involve many sites (i.e., many miles or highways and merged accident data) and/or multiple years of accident data. Increasing the number of study sites or the number of years of accident data, however, introduces other problems (i.e., cost) and fails to overcome the location and subset limitations previously discussed. Thus, it will be extremely difficult to develop the desired relationships by using multiple sites and/or years of data to predict accident frequency, rate, and/or types. A different approach is needed to study and/or identify such a relationship.

Limited Sight Distance Accident Studies

To date, three studies have been conducted with the objective of examining the relationship between accident frequency and stopping sight distance. The first of these studies is documented in NCHRP 270 (*2*) and is based on a matched pair comparison of accident rates on crest vertical curves in Michigan. The second study is documented in a Texas Transportation Institute (TTI) research report (*3*) that used multiple regression analysis to analyze accident rates and available sight distance at crest vertical curves in Texas. The third study, described in a 1991 issue of Public Roads (*4*), used the HSIS data base to analyze accident rates at crest vertical curves in Utah. Summaries of these studies follow.

Michigan Study. In the Michigan study (*2*), ten crest vertical curves with limited stopping sight distance (118 to 308 feet) were paired with ten nearby curves with adequate stopping sight distance (greater than 700 feet for nine of the ten control curves). The 20 curves in the study ranged from 0.15 miles to 0.50 miles in length; however, the paired curves within each study segment were of equal length. Paired curves were also matched in terms of traffic volumes, abutting land use, vegetation, road geometry, lane widths, and shoulders. Five year accident histories were obtained for each of the 20 curves in the study.

Figure F-1 shows the number of accidents that occurred at each of the ten pairs of curves. Note that if crest vertical curves with limited sight distance were no more hazardous than crest vertical curves with adequate sight distance, all ten data points in Figure F-1 should have fallen along the heavy diagonal line (plus or minus chance variation). The fact that most of the data points are above the diagonal line, suggests that relative to their matched control site, the curves with limited sight distance were over represented in high accident locations.

The solid line above the heavy diagonal line in Figure F-1 represents *the weighted average effect of limited sight distance on accidents*, based upon these ten data points. This *regression* function passes through the origin and has a slope of 1.435, i.e., accidents at the ten curves with limited sight distance exceed the expected number of accidents at the 10 curves with adequate sight distance by 43.5 percent. The Michigan study (*2*), estimated that the ten limited sight

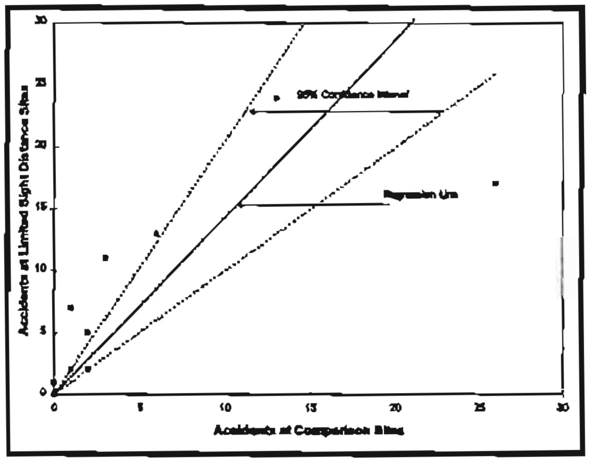

FIGURE F-1. The Relationship Between Accidents at Sites with Limited Stopping Sight Distance and Comparison Sites with Adequate Stopping Sight Distance (2).

distance curves had 51.9 percent more accidents than ex pected; however, as documented in other publications (*5, 6*), the technique used to arrive at this estimate (51.9 percent increase in accidents) is biased. The rationale for the 43.5 percent estimated increase in accidents is provided in the same publications.

This 43.5 percent estimated increase in accidents is significant at the 95 percent confidence level; however, the test statistic is barely in the rejection region. More important, the 95 percent confidence interval for the estimated increase in accidents has a wide range—from a 0.2 percent increase in accidents to a 105.6 percent increase in accidents. That is, the slope of the upper and lower solid boundary lines in Figure F-1 are 1.002 and 2.056, respectively. In other words, we are reasonably certain that the increase in accidents was somewhere between 0.0 and 100.0 percent.

The number of accidents occurring at each site during the five-year study period is shown in Table F-1. The accidents at Site Pairs 1 through 6 occurred from 1977 to 1982, whereas the accidents at Site Pairs 8 through 10 occurred from 1978 to 1983 (*2*). The accidents for Site Pair 7 were recorded from 1977 to 1981 (four years) because the roadway was widened in 1982. At seven of the site pairs, the curve with limited sight distance had more accidents; at two of the site pairs, the curve with limited sight distance had the same number of accidents; and at one of the site pairs, the curve with limited sight distance had fewer accidents.

Thus, in nine of the ten site pairs, the number of accidents at the limited stopping sight distance site had the same or more accidents than the corresponding control site with adequate stopping sight distance. As mentioned, this difference was statistically significant and led the Michigan researchers to conclude that limited stopping sight distance resulted in increased accident rates; however, we believe that this conclusion is somewhat misleading. Note that, with one exception in Table F-1, all of the limited sight distance sites had available stopping sight distance of less than 300 feet. Thus, a more accurate conclusion would be that **vertical curves with less than 300 feet of stopping sight distance resulted in increased accident rates**.

Texas Study. In the TTI study (*3*), accident and roadway data from 222 Texas highway segments, each approximately one mile in length, were collected and analyzed. Collectively 1,500 accidents occurred at these segments during a four-year period. For each highway segment in the Texas study, several descriptive variables were available: traffic volumes (1,500 to 6,000 vehicles per day), intersecting roads (county roads and/or numbered highways), cross roads influenced by limited sight distance, and the percent of the segment with stopping sight distance below a certain length. It was this last measure that is of primary interest to this study.

The Texas study's basic hypothesis was that accident frequency or rate at sites with high percentages of limited stopping sight distance would exceed the accident frequencies or rates at sites with lesser percentages of limited stopping sight distance (i.e., accident rates were a function of the amount of limited stopping sight distance on the segment). When log accident frequency (accidents/mile/year) and log accident rates (accidents/million vehicle miles) were regressed on these variables, the percent of the segment with limited sight distance was not a significant contributor to the regression model. Thus, it was concluded that **limited stopping sight distance had no discernible effect on accident frequency or rate** at the 222 Texas sites.

To better appreciate just how *noisy* and easily misinterpreted accident data can be for estimating safety effects of limited stopping sight distance, consider the data in Figure F-2. In this figure, 54 of the 222 highway segments in the Texas study are shown. Each of these 54 data points represents a segment of two-lane highway without shoulders. The vertical axis is *Accidents per Mile per Year* and the horizontal axis is the percent of highway segment with a *stopping sight distance of less than 450 feet*. If there is any pattern to these data points, it suggests that accident rates **decrease** as the amount of limited stopping sight distance **increases**; however, intuitively we know that increasing the amount of limited stopping sight distance should not enhance safety. In this data set, other variables that affect accident rates are undoubtably giving the false impression that increasing limited stopping sight distance may be beneficial.

As stated earlier, when these data were analyzed using standard multiple regression techniques, the amount of limited stopping sight distance had no discernible effect on accident rates. The one noticeable effect was an increase in accident rates when an intersection was located near the crest of the curve. These findings suggest that either the safety effects of limited stopping sight distance were not of sufficient strength to be detected by the regression analysis or the available sight distance at the study sites was in the range that has no effect on safety. Most of the vertical curves in the Texas study had minimum sight distances above 325 feet.

TABLE F-1. Description of Pairs of Sites (*2*).

Site Pair	Site Type	Minimum Available Stopping Sight Distance (feet)	Number of Accidents for Five Years
1	LSD[1]	118	11
	Control	> 700	3
2	LSD	276	1
	Control	536	0
3	LSD	188	2
	Control	> 700	2
4	LSD	174	7
	Control	> 700	1
5	LSD	263	13
	Control	> 700	6
6	LSD	250	17
	Control	> 700	26
7	LSD	262	24
	Control	> 700	13
8	LSD	308	5
	Control	> 700	2
9	LSD	280	2
	Control	> 700	1
10	LSD	223	0
	Control	> 700	0

[1] *LSD = Limited Sight Distance*

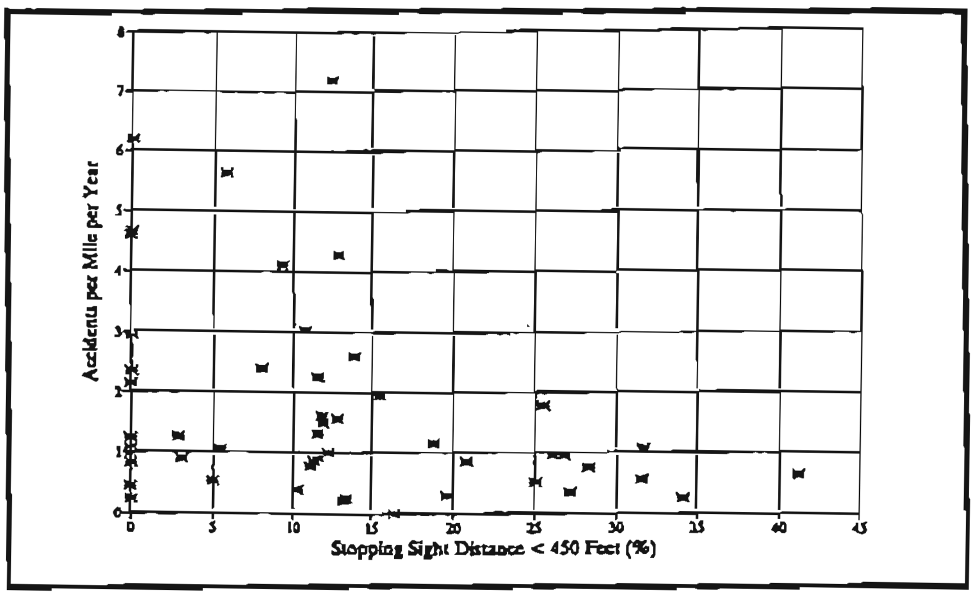

FIGURE F-2. Accidents per Mile per Year in Limited Sight Distance Sections on Two-Lane Roads without Shoulders (*3*).

Utah Study. The Utah study used accident data from 2,396 crest vertical curves in the HSIS data base (*4*) to analyze the relationship between vertical curve geometry and safety. Utah was chosen for study because it was the only state in the HSIS data base with complete information on vertical curve approach grades and lengths. Illinois could not be used because it only had information on substandard vertical curves, and the other three states had no information on vertical curve locations.

Three-year accident histories were merged with each of the 2,396 crest vertical curves in terms of distance from the crest to the reported accident locations. For each curve location, accident totals were produced for each 100-foot interval in both directions from the curve's crest. To remove the possibility of adjacent vertical curves influencing the accident experience at the vertical curve of interest, curves with less than 800 feet of separation between approach grades were omitted from the data set. After this check was made, the remaining data set contained 1,424 crest vertical curves.

The number of accidents in each distance interval was divided by the number of vertical curve locations to produce the number of accidents per crest vertical curve location. These accident rates per distance from the crest were plotted for curves with grade differences of 1 to 3 percent, 3 to 6 percent, and greater than 6 percent; however, because there was virtually no difference in the first two categories, they were subsequently combined. The results of this analysis are illustrated in Figure F-3 on the previous page.

As shown, accident rates are higher near the crest of the vertical curve, then level off to a relatively constant rate between 100 and 400 feet from the crest. Higher accident rates were seen for the curves with grade differentials greater than 6 percent, but the difference is most noticeable in the first 100-foot interval from the crest. In other words, more accidents occurred within the 200-foot interval centered at the middle of the crest vertical, and greater changes in grades are associated with more accidents near the crest of the vertical curve.

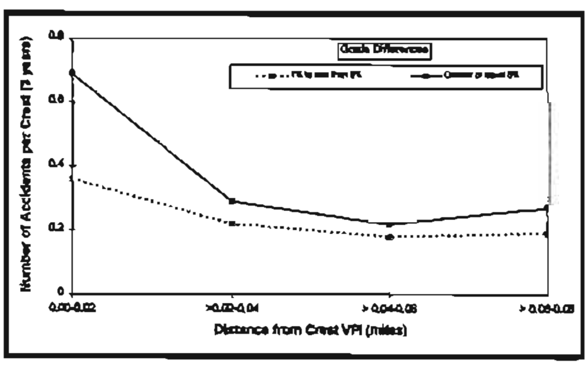

FIGURE F-3. Accident Frequency Relative to Distance from Crest Vertical Curve (*4*).

One of the assumptions of this analysis was that greater grade differential and shorter grades were equivalent to shorter stopping sight distances; however, the actual stopping sight distances could not be estimated because vertical curve lengths were not available in the Utah data base. Thus, there was a noticeable difference in accident rates because of grade differentials and distance from the curve's crest, but without knowing the curve lengths and resultant stopping sight distances, it is not clear whether this difference was the result of available sight distance, steepness of grade, or some combination of the two. This makes it impossible to establish a relationship between stopping sight distance and safety using the Utah data.

RELATED SIGHT DISTANCE STUDIES

The following sections describe three studies related to stopping sight distance and safety. The first study investigated large truck accidents and their causal factors; the second study investigated object-related accidents and their characteristics; and the third study investigated the relationship between operating speed and available stopping sight distance.

Large Truck Accidents. The American Automobile Association (AAA) Foundation for Highway Safety has a data base containing information on 231 large truck accidents that occurred between 1983 and 1984 on Interstate Highways in six western states (*7*). Inadequate or limited stopping sight distance was not cited as a contributing factor in any of the 231 accidents. There also was no indication from the factors contributing to the accident that the roadway's geometry limited the driver's view of the roadway. This result was not totally unexpected as Interstates have relatively few locations with limited stopping sight distance.

Only six of the 231 accidents involved objects or animals in the roadway. None of the six object-related accidents resulted in a fatality. One object was not identified, the second object was a parked car, and four objects were animals: two cows, one deer, and one porcupine. Thus, four of the five objects identified were not representative of the stopping sight distance situation, i.e., accidents involving small objects. The four accidents involving large objects occurred at night, and the one accident involving a small object occurred during the day; however, in this particular accident, it was raining and the pavement was wet. The truck's driver swerved to miss the porcupine and hit a bridge abutment. In none of these five accidents was there any indication that the roadway was other than straight and level.

In addition to the AAA data base, all 1,990 single vehicle accidents involving large trucks in Texas were reviewed in-house to ascertain the extent to which limited stopping sight distance might be a contributing factor to large truck accidents. Of the 2,230 single vehicle truck accidents identified, 14 (0.6 percent) occurred on hill crests (that may or may not have had adequate stopping sight distance), with 8 (0.4 percent) of the 14 accidents occurring during the day when roadway geometry rather than vehicle headlights limit available sight distance. Thus, even if limited sight distance was a contributing factor in all 8 daytime accidents, it contributed to a very small number of large truck accidents.

Object-Related Accident Studies. Kahl (*8*) examined object-related accident reports and narratives from two states to determine the type and size of objects struck in accidents that might be related to stopping sight distance. Three types of accidents were evaluated: other object accidents, animal accidents, and evasive action accidents. It was concluded that:

- Two percent of all reported accidents involved objects or animals on the roadway, and only 0.07 percent of all reported accidents involved objects or animals less than six inches (150 mm) high. Thus, small objects were not struck often enough to justify their use in the stopping sight distance model.
- More than 90 percent of the object and animal related accidents occurred on straight, level roads where the driver's visibility was not limited by the geometry of the roadway. Thus, the available stopping sight distance was not a major contributory factor in object and animal related accidents.
- Most of the object and animal related accidents occurred at night when the driver's visibility is limited by their vehicle's headlights. Because headlight sight distances are generally less than required stopping sight distances, longer crest vertical curves will not necessarily increase the driver's available stopping sight distance at night.
- More than 95 percent of the object and animal related accidents resulted in low severity accidents. Thus, a small object does not represent a hazard to most drivers.

Kervirtis (*9*) studied driver behavior when confronted with an unexpected object in the roadway. He concluded that when drivers are given the choice to stop completely, go around, or pass over an object in the roadway, they almost all choose to pass over objects up to four inches (100 mm) in height; they are equally likely to pass over or go around objects six inches (150 mm) in height; and they almost all choose to go around objects more than eight inches (200 mm) in height. These findings suggest that **drivers do not perceive small objects as hazards** and would help explain why there are so few reported accidents with small objects on the roadway.

Summary

The previous sections documented the difficulty in quantifying the relationship between safety and stopping sight distance. The conclusion from the Michigan study was that limited stopping sight distance creates safety problems, and the conclusion from the Texas study was that limited stopping sight distance did not create a safety problem; i.e., increasing the amount of limited stopping sight distance did not increase accident rates. At first glance, these results appear contradictory and confusing; however, collectively they point out that one range of limited stopping sight distances created safety problems and another range of stopping sight distances did not create safety problems. In other words, if the available stopping sight distance was marginally less than the AASHTO recommended value, it appeared to have no effect on accident rates; however, once the available sight distance was less than some *threshold value*, stopping sight distance was truly limited and it appeared to have an adverse effect on accident rates.

In the Michigan study, Olsen et al.(*2*) concluded that vertical curves with stopping sight distances less than 310 feet had a higher number of accidents than vertical curves with very long stopping sight distances. The Michigan data also suggested that the largest increase in accidents occurred at the study sites which had the shortest stopping sight distances. In the Texas study, Fambro et al. (*3*) concluded that in the range of 325 to 450 feet, limited stopping sight distance had no discernable effect on accident rates unless there was an intersection within the limited sight distance section. This finding implies that within this range, there are no safety benefits from providing additional stopping sight distance unless there is a hazard within the limited sight distance section. It is also consistent with Glennon's conclusion that alignment changes may be cost-effective only on high traffic volume highways with major hazards (such as intersections and sharp curves) within the limited sight distance section (*10*).

Conceptually, the relationship between stopping sight distance and safety at crest vertical curves can be illustrated as shown in Figure F-4. Accident rates increase for short sight distances and are relatively insensitive to sight distances above some *threshold value(s)*. That is, there is a wide range of sight distances that satisfy driver needs for safety and have similar accident rates. Additional hazards within the limited sight distance section may affect this threshold value by shifting the curve to the right. The challenge for researchers and practitioners is defining the point and/or conditions at which accident rates begin to increase.

FIGURE F-4. Conceptual Relationship Between Available Sight Distances and Safety at Crest Vertical Curves.

METHODOLOGY

The previous section presented a conceptual model of the relationship between stopping sight distance and safety, and pointed out the difficulty in using existing accident data bases to develop such a relationship. The methodology selected for this research was a case study approach wherein a relatively small sample of accidents on roadways with limited stopping sight distance was chosen and an in-depth analysis of each accident was conducted. The analysis concentrated on identifying patterns of contributing factors associated with accidents on roadways with limited stopping sight distance. The rationale for selecting this approach was that if limited stopping sight distance affected accident rates, it should be a frequent contributing factor to accidents on roadways with limited stopping sight distance.

This section, *Methodology*, which documents the procedures used in the accident causation study, is organized in the following manner: case study approach, data-collection procedures, accident narrative report analysis, data analysis, and site-specific information. The final section presents a summary description of the 37 limited stopping sight distance sites selected for the accident study.

Case Study Approach

The possibility that stopping sight distance is a contributing factor in some accidents cannot be ruled out just because a relationship between stopping sight distance and safety has not been established with existing accident data bases. That is, we know that limited stopping sight distance has to result in safety problems at some point. The approach selected for this research was an in-depth diagnostic case study to analyze the accidents which occurred on a sample of roadways containing limited sight distance crest vertical curves. The diagnostic case study involved analyzing accident narratives for selected sites to identify accident patterns, and/or common characteristics that might be related to limited stopping.

As stated in the literature review, previous studies attempting to relate stopping sight distance to safety have been inconclusive and inconsistent. These previous research efforts were concerned with statistical comparisons of the resulting accident experience rather than analyzing the contributing factors and physical features of the accident sites. By performing a diagnostic case study of the accidents at the selected study sites (i.e., accident reconstruction), the roadway conditions/patterns conducive to having limited stopping sight distance be a contributing factor to an accident can be determined.

The process used in the case study approach for this research is shown in Figure F-5. Note that the process began with identifying a sample of sites with known limited sight distance curves. Police officers' narratives of the accidents that occurred at these sites were then examined to determine the contributing factors to the accidents. The findings from those examinations and the geometric characteristics of the sites were used to identify accidents in which limited stopping sight distance was a possible contributor to the accident.

Data Collection Procedures

Because roadway type and traffic volume may influence accident rates, the classification scheme shown in Table F-2 was developed to aid in the selection of potential study sites. Roadway types included freeways, multilane highways, two-lane with shoulder roadways, and two-lane without shoulder roadways. Roadways were categorized as having shoulders if the paved shoulder was six feet or wider and without shoulders if the shoulder was unpaved or less than a six-foot in width. Traffic volume levels were divided into low, medium, and high, but the exact threshold values varied by type of roadway.

Note that eight sites (four sites in each of two volume categories) were planned for each of the four roadway types. Note also that no study sites were planned for the low volume/high design or high volume/low design categories. These combinations of conditions were not likely to be found in the field. Additionally, it was expected that limited sight distance study sites on freeways and multilane highways would be difficult to locate because of the generally higher design standards on these types of roadways.

Site Identification. In order to represent the range of conditions present in the driving environment, potential sites were identified in three geographic regions of the United States— Washington, Texas, and Illinois. These states were selected because they contain rolling terrain, where limited sight distance crest vertical curves are generally found, and they have sufficient roadway geometry data bases to identify limited sight distance curves. For example, Texas and Washington have computerized roadway geometry and accident data bases, and the Illinois roadway geometry and accident data are contained in the Highway Safety and Information Systems (HSIS) computer data base. The following criteria were developed to identify potential accident study sites:

- The potential study site should be classified as rural.
- The posted speed limit along the entire length of the study site, including horizontal curves, should be 65 mph (105 km/h) for freeways and 55 mph (88 km/h) for multilane and two-lane roadways.
- The study site should not be within 0.5 miles (0.8 km) of a signalized intersection.
- The study site should not contain any major intersections (e.g., an intersection with a numbered highway).
- The study site should not contain any other geometric or roadside feature, such as a narrow bridge, which would decrease the desired operating speed or increase the accident experience.

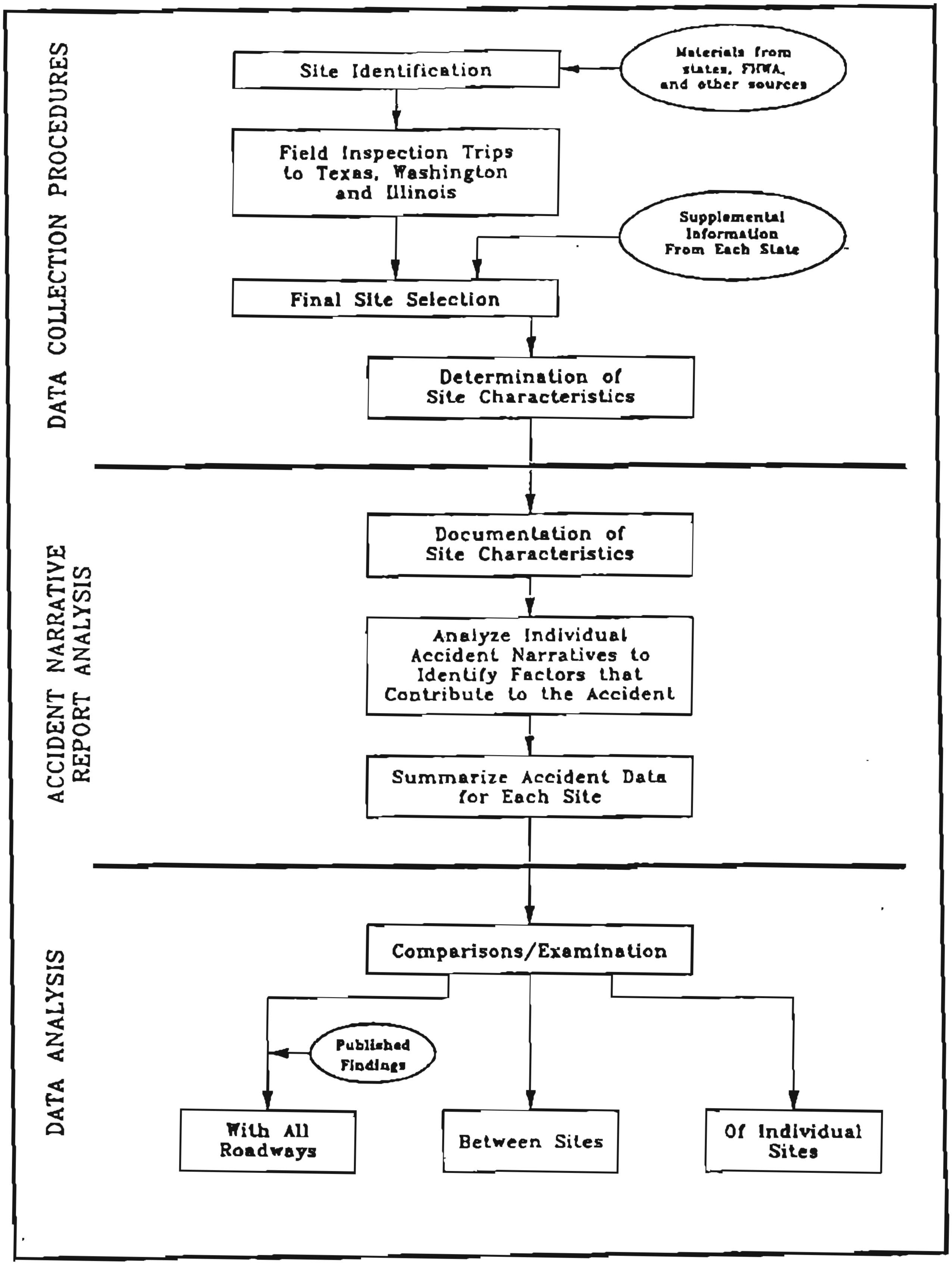

FIGURE F-5. Flow Chart of Accident Causation Study.

TABLE F-2. Number of Sites by Roadway and Traffic Conditions for Accident Studies.

Type of Roadway	Traffic Volume Levels			Total
	Low	Medium	High	
Freeway	0	4	4	8
Multilane	0	4	4	8
Two-Lane with Shoulders	4	4	0	8
Two-Lane without Shoulders	4	4	0	8
Total	8	16	8	32

- The study site should contain at least three limited sight distance crest vertical curves and no major intersections or horizontal curves. A crest vertical curve was classified as limited if it had less than 450 feet of available sight distance (550 feet for a freeway). The 450-foot value is the minimum stopping sight distance for a 55 mph design speed, and 550 feet is the minimum stopping sight distance for a 65 mph design speed (*11*).

- The study site should be between one and five miles in length.

The rationale for these criteria was that signalized intersections, major intersections, and varying speed limits are factors which contribute to increased accident rates. Accidents caused by these factors may be so numerous as to overwhelm and mask identification of those accidents with limited stopping sight distance as a contributing factor. Therefore, attempts were made to eliminate or minimize the influence of these factors through the site selection process. Desirably, the limited sight distance crest vertical curves were separated by less than one-half mile; however, spacings up to one mile between consecutive limited crest vertical curves were allowed.

Because so few sites with the above criteria were identified, the requirement of vertical curves not separated by a horizontal curve or a major intersection was relaxed to include sites where vertical curves were separated by horizontal curves with long radii, which would not decrease the operating speed. Sites on long tangents with a single limited sight distance vertical curve were also included as potential accident sites.

The beginning and end points of each accident study site were selected as one-half mile before the vertical point of intersection (VPI) of the first limited sight distance curve in the segment and one-half mile beyond the VPI of the last limited sight distance curve in the segment. The reason that the accident site boundaries were extended beyond the limited sight distance crest vertical curve was due to the uncertainty of the exact accident location, described in the previous section. For several of the accident study sites, the one-half mile range beyond the vertical point of intersection (VPI) of the last limited sight distance crest vertical curve had to be decreased, due to a limiting geometric feature such as a nearby intersection or horizontal curve.

Field Inspection Trips. After a list of potential sites was developed, field trips were made to the three states to visually inspect each site. The same team of two individuals participated in each trip. The sites were inspected in the field to verify that the field conditions represented the conditions identified in the office from the roadway data bases. The field trips also provided the opportunity to identify characteristics of the sites not available from any other source.

For example, during the visits, each potential accident site was videotaped and photographed to provide a permanent record of the roadway geometry and site specific characteristics of that site. The width of the travel lanes, width and type of shoulders, and features of the clear zone were also documented. In addition, the number and location of driveways, intersections, median openings, and bridges were recorded.

Final Site Selection. Several potential sites were eliminated or study limits redefined during the site inspection trips. Based upon the findings in the field, a feasible site list was developed for each state. Supplemental information was requested from the appropriate agencies for these feasible sites. This information included plan and profile drawings, traffic volumes at each site, and the accident narrative reports.

The plan and profile drawings provided detailed information on the geometry at each site and also provided an overall perspective of the entire roadway segment. The Average Annual Daily Traffic (AADT) values for each site were needed so that the average accident rates could be determined. The AADT values for the Illinois and Washington sites were requested from the respective states. The AADT values for the Texas sites were obtained from the State's computerized accident data base.

Each state was asked to indicate the number of accidents occurring at each of the potential study sites during the previous three- and five-year periods. The decision on the number of years to actually request from each state was based on the number and cost of obtaining the accident reports from the state. The accident narratives requested were the previous five years (April 1988 to March 1993) from Washington, the previous three years (January 1990 to December 1992) from

Illinois, and the most recent three years available (April 1990 to March 1993) from Texas.

Determination of Site Characteristics. Each site's vertical and horizontal geometry was sketched to determine appropriate beginning and ending points for the study site. These points were generally set one-half mile beyond the first and last crest vertical curves in the study segment. The one-half mile distance was decreased if it included a major intersection, a horizontal curve, or other element which was believed to have a significant influence on accidents. When consecutive limited sight distance crest vertical curves were separated by distances greater than one mile, the segment was split into two sites so that each curve could be analyzed separately.

The reason for the split was due to the travel time between the limited stopping sight distance curves. At 55 mph (88 km/h), a driver would encoumter a limited sight distance curve approximately once every 63 seconds if the curves were spaced at exactly one mile. The effect of consecutive limited stopping sight distance curves spaced at this distance is thought to be far enough apart to treat the two curves as separate study sites; i.e., there is no carryover effect from one limited stopping sight distance curve to another.

Once the limits of each site were selected, the number of approaches—both intersections and driveways—and the number of limited sight distance curves greater than 100 feet in length were counted. Crest vertical curves shorter than 100 feet were not included because short vertical curve lengths do not restrict the driver's view of the roadway ahead; i.e., the driver can see over the vertical curve crest. The lane width, shoulder width, and average daily traffic for the site were also identified from the roadway data files.

Due to the diversity of the selected accident sites, a "subjective rating" was developed to classify the sites with similar vertical alignments. The "subjective ratings" were based upon the following considerations:

- Number of limited sight distance crest vertical curves within the site;
- Distances between consecutive limited sight distance crest vertical curves;
- Percentage of limited sight distance crest vertical curves which had upgrades followed by downgrades (defined by AASHTO as "Type I" vertical curves). Crest vertical curves which had upgrades followed by flatter upgrades or downgrades followed by steeper downgrades are defined as Type II vertical curves and are generally less of a sight distance limitation;
- Consistency of available sight distance values for the limited sight distance crest vertical curves located within the study limits.

A subjective rating of "1" indicated an isolated Type I crest vertical curve with limited sight distance. A subjective rating of "3" indicated closely-spaced Type I crest vertical curves, each with similar values of minimum stopping sight distance. Sites with a subjective rating of "3" generally had a *roller coaster* profile. Finally, a subjective rating of "2" indicated a site which could not be classified as "1" or "3". Generally, a site with a subjective rating of "2" contained limited sight distance curves which were spaced 0.5 miles or farther apart, contained several Type II limited sight distance crest vertical curves, or contained limited sight distance crest vertical curves with a range of minimum stopping sight distance values.

Accident Narrative Report Analysis

The following procedures were used to ensure consistent evaluation of all accidents that occurred at the study sites. The specific steps that were followed include documentation of site characteristics, analysis of individual accident narratives, and a summary of the data at each site.

Documentation of Site Characteristics. Before an in-depth accident analysis was performed, the geometry and roadside characteristics of each site were assembled and used in the preparation of a straight-line diagram. The straight-line diagram was used to locate all accidents by sequential milepoint along the route. Accident report numbers and milepoints were identified on the straight-line diagram, along with horizontal and vertical curves and major intersections. Figure F-6 is an example of a typical straight-line diagram.

Next, the two individuals reviewing the accident narratives supplemented the information from the straight line diagram by viewing the video tape and slides of the sites that were made during the site inspection trip. The video tape was used to identify both geometric features and site specific features, including type and presence of shoulder, type of ditch, width of clear zone, and location of major intersections, driveways, and other limiting geometric features. The plan and profile drawings also were reviewed to identify the locations of site-specific features.

Analyze Individual Accident Narratives. An *accident summary form* was developed to ensure consistent evaluations of the accident narrative reports reviewed for this study. General information, including milepoint, route, date, time, pavement and weather conditions, was recorded on the form. After reading the accident narrative, the sequence of events that occurred during the accident, as determined by the two researchers reviewing the accidents, was recorded on the *accident summary form*. A sample *accident summary form* is shown in Figure F-7. Due to the confidentiality of accident reports, the routes, accident milepoints and report numbers are not identified.

For each event in the sequence, the primary and secondary contributing factors (if any) were identified. From Figure F-7, the contributing factor, *glare*, identified as "1", was the primary contributing factor to the accident. If there were other contributing factors in this accident, they would have

Straight Line Diagram
Washington State
SR XX

Milepoint	Accident Report Number
33.0	
	88-119811, 92-221134
33.5	
VC	*90-456725, 91-653345*
	91-583468
34.0	
VC	
	88-843621, 89-589213
	92-859323, 91-658414
34.5	
	92-885325, 92-978877, 91-584414
HC	
35.0	
	89-655358, 89-655396
VC	
Inter	*88-859936, 90-541121, 92-877436, 92-544496, 90-655258*
35.5	
	92-844695, 88-522395

FIGURE F-6. Straight Line Diagram.

Accident Summary

State Texas

Report Number

Route

MP

Weather Clear

Pavement Dry

Date 1/14/92

Time of Day Day

Time 16:00

Available Light Daylight

Geometry Private Drive

Sequence of Events

1. Vehicle #1 and Vehicle #2 southbound.
   Contributing factors
2. Vehicle #1 slowed to make right turn into private drive.
   Contributing factors
3. Vehicle #2 struck Vehicle #1.
   Contributing factors Glare (other)[1]
4.
   Contributing factors
5.
   Contributing factors
6.
   Contributing factors

Assumptions

1. None
2.
3.
4.

Vehicle Damage Over $500

Occupant injury severity None

Level of Confidence 1 2 3 4 **5**

Low High

Sight distance is not a contributing factor.

FIGURE F-6. Accident Summary Form.

TABLE F-3. Contributing Factors to the Accident.

Sight distance	Limited geometry - vertical curve
Headlight sight distance	Temporary roadside activity
Intersection sight distance (horizontal curve)	Speed too fast for conditions
Intersection sight distance (vertical curve)	Unexpected object in the roadway
Pavement condition	In car distraction
Weather	Driver intoxication
Snow/ice pavement	Driver fell asleep
Wet pavement	Driver negligence
Limited geometry - horizontal curve	Other

been identified as 2, 3, 4, etc. The contributing factors used in the analysis are listed on Table F-3. In order to limit the number of contributing factors that were used, the other factor category was used to group those factors which were not used extensively.

For some of the narratives, portions of the desirable information for reconstructing the accident were missing. Therefore, based upon the information available on the accident narrative, a *level of confidence* was determined for each accident in the data base. For example, if several factors had to be assumed, such as in a hit-and-run accident where driver statements were not available, the narrative review would receive a low *level of confidence*.

Summarize Accident Data for Each Site. After all accidents at a study site were evaluated, a *contributing factor summary* was developed for the site. This summary sheet included the sequential order in which the accidents occurred, milepoints, lengths, and K-value or degree of curve, for vertical and horizontal curves, respectively. Also included in the summary was an indication of whether the accident involved a younger (20 years or less) or older (at least 55 years) driver and whether it involved a large truck. A sample *contributing factor summary* is shown in Figure F-8.

Each row in Figure F-8 represents an accident that occurred on the roadway. The columns represent the contributing factors for each accident. For example, the second accident occurred at milepoint 5.7, and is identified in the second row. Contributing factor 12 (speed too fast for conditions), identified by *the number 1*, was the primary contributing factor, and contributing factor 8 (wet pavement), identified by *the number 2*, was the secondary contributing factor. From these summary sheets, trends or common characteristics of factors contributing to accidents on roadways with limited sight distance vertical curves can be determined.

Data Analysis

The data analysis was divided into three areas—comparison of the limited sight distance sites with other (non limited) sites, comparison of different groupings of the limited sight distance sites, and an examination of individual accidents. These steps are described in the following sections.

Comparison of Limited Sight Distance Sites with Other Sites. As mentioned, a case study approach was selected for this study because previous studies suggested that a comparative study would not be successful; however, limited comparisons of the accident rates for sites in this study with the accident rates for a large sample of other sites will provide some insight into whether limited sight distance sites differ from the overall roadway system in terms of accident experience.

Initially, the data were separated by roadway type (i.e., two-lanes without shoulders, two-lanes with shoulders, multilane highways, and freeways) and compared to published values. The accident rates and the percent of older/younger and tractor-trailer accidents also were calculated and compared to published values. Noticeable differences between the accidents rates on limited sight distance sites and the general roadway population would indicate areas for additional investigation.

Comparison Between Limited Sight Distance Sites. Comparison between different groups of sites could indicate whether certain features are major contributing factors to accident occurrence. For example, if the two-lane sites with shoulders have consistently lower accident rates than the two-lane sites without shoulders (assuming similar types of limited sight distance curves and other characteristics), one might conclude that the absence of shoulders is a contributing factor to accidents at limited stopping sight distance curves. Generally, the comparisons were conducted by plotting the accident rates (accidents per million vehicle miles and accidents per year per mile) by the categories of interest (i.e., roadway type, state, and design speed). The following between-group comparisons were performed:

- Accidents per approach;
- Classification by roadway type (multilane, two-lane with shoulders, two-lane without shoulders);
- Classification by state;
- Classification by mean design speed of all Type I limited sight distance crest vertical curves within the segment; and

Contributing Factors
Texas Route XX

Multilane
High ADT (14,000)

Acc MP	Curve	Len (feet)	Crv MP	K or D	Age	Contributing Factors: TT	1 2 3	4 5 6	7 8 9	10 11 12	13 14 15	16 17 18
5.7											1	
5.7									2	1		
5.7											1	
5.7											1	
5.7											1	
5.8												1
5.8						X			1	2		
5.8					YO				1	2		
6.0					Y					1		
6.0	Crest	792	6.1	92			2				1	
6.1									2	1		
6.1					Y				1	2		
6.1												1

Contributing Factors
1=Sight distance
2=Headlight sight distance
3=Intersection sight distance (HC)
4=Intersection sight distance (VC)
5=Pavement condition
6=Weather
7=Snow/ice pavement
8=Wet pavement
9=Limited geometry (HC)
10=Limited geometry (VC)
11=Temporary roadside activity
12=Speed too fast for conditions
13=Unexpected object
14=In car distraction
15=Driver intoxicated
16=Driver fell asleep
17=Driver negligence
18=Other

Age
Y=Younger (20 or younger)
O=Older (55 or older)

TT=Large truck

FIGURE F-8. Contributing Factor Sheet.

- Classification by the critical (or minimum) design speed of all Type I limited sight distance crest vertical curves within the segment.

After plotting the resultant graphs for each roadway type, different site classifications were developed to explain the accident rate variation within each subjective rating category.

Examination of Individual Accidents. The *contributing factor summary* developed for each site provides an overview of the different elements that contributed to each accident at each site. The contributing factors for all sites were summed by roadway classification. The most frequent contributing factors for each roadway classification were targeted for additional discussion.

Additional effort was expended on those contributing factors that are believed to be related to stopping sight distance. Those contributing factors included sight distance—either during daytime or nighttime and unexpected objects in the road. Accidents involving one of those contributing factors were reexamined to determine if similar characteristics (e.g., no shoulders) or driving environment (e.g., nighttime or icy conditions) existed between the different sites. This type of examination also could be used to suggest low-cost countermeasures to improve safety that did not involve reconstructing the limited stopping sight distance vertical curve.

Site Information

The site specific information describes specific procedures and sites selected in each of the three states—Texas, Washington, and Illinois. Roadway and traffic data for each of the selected study sites are also included in this section.

Texas. Roadway plans and detailed geometric information were available from several previous projects. This information was used to provide an initial list of potential sites for this study. Phone conversations with representatives of districts in Texas that have rolling terrain and older roads (and therefore, presumably sites with limited sight distance curves) and the knowledge and research team's familiarity with Texas roads also provided suggestions of locations that may have limited sight distance curves.

Each potential site was inspected in the field along with any site identified during other project trips. Sites were eliminated if the site was not in a rural area, or had a major intersection, horizontal curve, or other feature that could greatly influence the number of accidents occurring. For each of the remaining feasible Texas sites, the location of driveways, intersections, bridges, and other relevant features were determined using a Distance Measuring Instrument (DMI). A DMI electronically determines distances with respect to the rotation of a vehicle's wheels.

Twelve sites, from the more than 40 Texas sites inspected, were selected for further analysis. Plan and profile drawings were requested for these 12 sites. During the review of the accident narratives, two of the sites were split into two separate sites so that an intersection and a horizontal curve could be eliminated from the study segment. This split resulted in 14 limited stopping sight distance study sites in Texas. Table F-4 summarizes the traffic and roadway geometry characteristics of the 14 sites.

Washington. Several sources of information were used to identify potential accident sites in Washington state. The Horizontal and Vertical Alignment Report contains information on curve locations by State Route Milepost (SRMP) and Accumulated Route Miles (ARM), horizontal curves (length, radius, and central angle), and vertical curves (approach grade, departure grade, and curve length). The 1992 State Highway Log Planning Report contains information about the number of lanes, shoulder types and widths, median types and widths, milepost locations of intersections and physical features, functional classifications, and legal speed limits. The 1991 Annual Traffic Report contained AADT values from 1988 to 1991.

The first step in site selection was to eliminate all non-rural sites and to eliminate all sites that had a speed limit of less than 55 mph (88 km/h) for two-lane and multilane roadways and 65 mph (105 km/h) for a freeway. Next, curves with less than 450 feet of sight distance were identified (550 feet for freeways). From this subset of limited stopping feet of sight distance vertical curves, potential accident sites were defined as a section with at least three successive limited stopping sight distance curves not separated by horizontal curves.

After classifying the sites based upon roadway type (freeway, multilane, two-lane with shoulders, or two-lane without shoulders) and AADT range (high, medium, or low), it was determined that the majority of the potential sites in Washington were two-lane roadways without shoulders. It also was determined from the State Highway Log Planning reports that there were no potential multilane sites because most multilane roadways with limited stopping sight distance were located in urban areas. Thus, the site selection criteria were modified in an attempt to identify additional multilane and two-lane with shoulder sites in Washington.

As a result of the need for additional study sites, criteria for potential sites were defined as three limited sight distance curves with one horizontal curve, or two limited sight distance curves not separated by any horizontal curves. Examination of the potential freeway sites revealed that the limited sight distance crest vertical curves were not close to one another. Due to the long distances between vertical curves, each curve was analyzed as an individual site. The boundary for each limited sight distance accident site was located 0.25 miles before the curve's VPI and 0.25 miles past the curve's VPI.

TABLE F-4. Accident Sites in Texas.

Route	Length (miles)	Appr/ Miles	1992 AADT	LSD Curves	Lane Width (feet)	Shoulder Width (feet)	Subject Rate
Multilane Roadways							
TX-A01	0.9	12.22	14000	1	12	0[b]	1
TX-A02	3.0	16.33	4375[a]	8	12		3
Two-Lane Roadways with Shoulders							
TX-A03	0.9	7.8	3300	1	12	10	1
TX-A04	3.0	9.0	4800	6	12	8	2
TX-A05	2.3	10.4	7600	3	12	8	2
Two-Lane Roadways without Shoulders							
TX-A06	3.8	22.89	2800	10	11	0	3
TX-A07	2.2	20.91	3600	4	12	0	2
TX-A08	2.7	31.48	5150	5	12	0	2
TX-A09	4.0	8.75	1000	4[c]	10.5	0	2
TX-A10	2.4	21.25	1500	3	11	0	2
TX-A11	1.2	14.17	3700	2	11	0	2
TX-A12	2.2	18.64	2000	4	10.5	0	2
TX-A13	2.8	13.93	940	10	10.5	0	2
TX-A14	2.3	11.30	940	2	11	4	1

[a] *1/2 of AADT, because of NB only*
[b] *1-foot paved, 3-foott gravel*
[c] *Excludes curves 100 feet or shorter*
Notes:

Appr/miles = *Number of approaches (intersections and driveways) per mile*
LSD Curves = *Number of limited sight distance curves*
Subj Rate = *Subjective rating of the vertical curve geometry — 1=isolated limited sight distance curve, 3=closely spaced limited sight distance curves, and 2=not 1 or 3*

All potential accident sites were located on a Washington state map. To determine their approximate location, the SRMP (State Route Milepoint) of the potential sites was compared to the SRMP of the intersecting roads, rivers, and/or towns. Upon inspection, it was determined that the majority of the sites were located in the western and central parts of Washington state. A priority system was developed to rank the potential sites since visiting all of them was not feasible. The sites that contained the fewest intersecting roads and were potential sites for an operating speed study, were given the highest priority for further study. Sites with an AADT less than 1000 vehicles per day were eliminated due to the low potential for accidents on these roadways.

A team of two individuals inspected each of the 20 potential accident sites and verified that the field conditions represented the conditions described in the Highway Log Planning Report. Four potential freeway sites were identified. Due to the large number of accidents on the freeway sites, they were divided into one-half mile segments (0.25 miles on either side of the vertical curve VPI). If two or more one-half mile segments overlapped, the segments were combined into one site. Table F-5 lists the roadway and volume data for the freeway sites. A total of 13 feasible non-freeway sites were located in Washington (see Table F-6).

The request for plan and profile data revealed that some of the Washington accident sites were asphalt overlays or bituminous surface treatments that followed the existing geometry. As a result, profile drawings were not available for these sites, and the Horizontal and Vertical Alignment Report was used to determine the geometry of the roadway. Those sites in which the profile drawings were available verified the information in the Horizontal and Vertical Alignment Report. Thus, there is no reason to suspect that not having the profile drawings for some of the Washington sites affected the quality of the roadway geometry data.

Illinois. Roadway geometry information for Illinois highways is contained in the HSIS data base. One of the variables stored in the data base indicates if a vertical curve had limited sight distance. FHWA provided a subset of the Illinois data for use in this study. The subset contained all records that represented a rural limited sight distance vertical curve. It included several variables, such as road name, county, milepoint, shoulder type and width, approach grade, departure grade, vertical curve length, and AADT values.

TABLE F-5. Freeway Accident Sites in Washington.

Route	Length (mile)	1991 AADT	LSD Curves	Lane Width (feet)	Shoulder Width (feet)	Number of Ramps
WA-A01	0.50	19200	1	12	10,6&10	0
	0.50	19300	1	12	10,?	0
	0.78	19300	2	12	10,4	0
	0.50	19300	1	12	10,4	2
WA-A02	0.50	17500	1	12	10,4	1
	0.50	18600	1	12	10,4	2
	0.50	20500	1	12	10,4	0
WA-A03	0.50	28300	1	12	10,4	2
	0.75	34000	2	12	10,4	3
	0.50	34000	1	12	10,4	3
	1.29	39400	4	12	10,4	5
	0.50	54700	1	12	10,4	1
	0.50	38800	1	12	10,4	2
	0.50	33800	1	12	10,4	2
WA-A04	0.42	5650	2	14	8,4	0

Notes: LSD Curves = Number of limited sight distance curves

TABLE F-6. Non-Freeway Accident Sites in Washington.

Route	Length (miles)	Appr/ Miles	1991 AADT	LSD Curves	Lane Width (feet)	Shoulder Width (feet)	Subject Rate
Two-Lane Roadways with Shoulders							
WA-A05	2.8	12.5	3275	6*	12	6	2
WA-A06	0.7	5.7	2742	2	12	6	1
WA-A07	0.8	5.0	3625	1	12	6	1
Two-Lane Roadways without Shoulders							
WA-A08	1.5	4.00	4150	3	11	4	2
WA-A09	0.9	23.33	2725	3	11	4	2
WA-A10	1.6	1.88	2750	3	11-14	3	2
WA-A11	1.9	3.16	2075	6*	12	3	2
WA-A12	1.5	4.67	6025	2	12.5	5	1
WA-A13	3.6	3.89	1545	11	10.6	4	3
WA-A14	2.0	2.00	1535	2	11.5	1-4	1
WA-A15	1.5	12.00	5200	4	12	4	1
WA-A16	1.7	1.76	800	5	10	4	2
WA-A17	1.5	23.33	4500	6*	12	3	3

**Excludes curves 100 feet or shorter.*

Notes:

Appr/miles = Number of approaches (intersections and driveways) per mile

LSD Curves = Number of limited sight distance curves

Subj Rate = Subjective rating of the vertical curve geometry — 1=isolated limited sight distance curves, 3=closely spaced limited sight distance curves, and 2=not 1 or 3

A program was written that would sort the Illinois data and identify sites with three vertical curves separated by less than one mile. Desirably, these curves were not separated by horizontal curves or major intersections. Detailed county maps were used to identify additional information about the site as the HSIS data base contained minimal information on location of intersections and horizonal curves. Because more sites were identified than could be inspected in a reasonable amount of time, sites that were located on a straight alignment and did not contain major intersections were assigned the highest priority, while sites with one or more horizontal curves or a major intersection were assigned a low priority. Additionally, multilane and two-lane roadways with shoulders were assigned a higher priority because the majority of the sites identified in the other states were two-lane roadways without shoulders. It should be noted that there were no limited sight distance vertical curves on Illinois freeways in the HSIS data base.

During the initial site visit, it was determined that only a few sites could be classified as "with shoulders" (paved six feet or greater). Several sites in the HSIS data base were listed as having shoulders greater than six feet in width; however, the shoulders were either stabilized grass or gravel. These sites were classified as without shoulder roadways because drivers may not feel comfortable driving on stabilized grass or gravel shoulders. Several of the sites were identified as questionable during the field inspection trip because the stopping sight distance at the crest vertical curves did not appear limited; however, these sites were retained in the list of feasible sites until the plan and profile drawings could be reviewed and the stopping sight distance verified. Seventeen of the 32 sites visited remained on the feasible list at the conclusion of the field trip.

The plan and profile drawings obtained after the site visit revealed some discrepancies between the plan and profile drawings and the information in the HSIS data base, which was used to select the potential sites. For some of the potential study sites, it appeared that the vertical grades were entered into the HSIS data base opposite from the grades on the profile drawings (i.e., a grade of +3% actually meant -3% and vice versa). Therefore, vertical curves identified as *crest curves* in the HSIS data base were identified as *sag curves* on the profile drawings. Additionally, it appears that the roadway at one or two of the other Illinois sites had been reconstructed, but the computerized data base had not been updated to reflect the reconstructed alignment and profile. The inconsistencies between the plan and profile drawings and the HSIS data base information resulted in elimination of 11 of the 17 Illinois sites because they did not contain limited sight distance vertical curves. The traffic and roadway geometry characteristics for the remaining six Illinois sites are illustrated in Table F-7.

TABLE F-7. Accident Sites in Illinois.

Route	County	Length (mile)	Appr/ Miles	1992 AADT	LSD Curves	Lane Width (feet)	Shoulder Width (feet)	Subject Rate
Two-Lane Roadways with Shoulders								
IL-A01	Washington	4.4	10.0	3450	10	12	8-10	3
Two-Lane Roadways without Shoulders								
IL-A02	Jo Daviess	1.7	5.88	5400	4	12	3-5	3
IL-A03	Stephenson	5.2	4.62	5200	17	12	3-5	3
IL-A04	McHenry	1.7	15.29	4500	5	11	1-3	3
IL-A05	Ogle	1.5	4.67	2400	3	12	4*	2
IL-A06	Ogle	3.1	8.39	1700	5	12	1	3

**1-foot paved, 3-foot gravel*

Notes: *Appr/mile* = *Number of approaches (intersections and driveways) per mile*
LSD Curves = *Number of limited sight distance curves*
Subj Rate = *Subjective rating of the vertical curve geometry — 1=isolated limited sight distance curves, 3=closely spaced limited sight distance curves, and 2=not 1 or 3*

Summary of Sites Selected

Approximately 100 sites were inspected in the field for possible inclusion in the accident study data base. Of those sites inspected, 46 sites passed the initial in-field screening; however, several of those sites were tentative until additional information on vertical curve geometry was available. Based on information gathered after the site inspection trips, 37 sites remained in the study's accident data base. The distribution of sites by the different states is shown in Table F-8.

When Table F-8 is compared to Table F-2, several differences between the number of sites proposed and the number of sites selected are evident. The original goal was to have eight freeway sites and eight multilane sites. After finding no freeway sites and only two multilane sites in the first state, identification of potential freeway and multilane sites in the other two states was given the highest priority; however, only four freeway sites and no additional multilane sites could be identified. In the third and final site inspection trip, additional emphasis also was placed on identifying two-lane sites with shoulders (at the expense of inspecting additional two-lane sites without shoulders).

Any multilane and two-lane with shoulder sites identified during the Illinois trip as possible limited stopping sight distance sites were included on the list of possible study sites. Several of these sites, however, were dropped afterwards because the vertical geometry information obtained from the plan and profile drawings indicated that the sites did not have limited sight distance curves. Having to eliminate most of the multilane and two-lane with shoulders sites in Illinois because they did not contain limited stopping sight distance curves is the primary reason for the low number of sites in these roadway type categories. Additionally, and as mentioned previously, limited stopping sight distance vertical curves are not as common on these roadway types.

More limited stopping sight distance sites on two-lane without shoulder roadways were identified than originally proposed—24 identified versus 8 proposed. These additional sites were near each other, facilitating data collection; and the presence of limited sight distance curves was more common on two-lane without shoulder roadways than on the other types of roadways. It also should be noted, however, that two-lane without shoulder roadways were the most common type of roadway in the three states in this study.

RESULTS

This section documents the findings from the accident study and is divided into three major areas: characteristics of limited sight distance sites, comparison between different groups of limited sight distance sites, and examination of individual accidents. *Characteristics* includes a comparison of limited sight distance sites to all rural sites and a summary of the characteristics of limited sight distance sites by presenting the percentage of younger and older driver accidents, the percentage of tractor-trailer accidents, and the percentage of accidents occurring during daylight conditions. This section also compares accident rates for the two-lane roadway sites with limited stopping sight distance to the overall accident rate for all two-lane rural roadways in two states.

Several different groupings such as geographic region, design speed, and roadway type were used to investigate differences in accident rates between the limited stopping sight distance sites in this study. These comparisons are described in the section entitled *Comparison*. The results of the case study, the major focus of this study, are contained in the section entitled *Examination*. This section includes descriptions and characteristics of accidents that might have been influenced by the presence of a limited sight distance curve. The final section presents a summary of the important findings from the accident causation study.

TABLE F-8. Number of Sites Selected for Accident Studies.

Type of Roadway	State			Total
	Texas	Washington	Illinois	
Freeway	0	4	0	4
Multilane	2	0	0	2
Two-Lane with Shoulders	3	3	1	7
Two-Lane without Shoulders	9	10	5	24
Total	14	17	6	37

Characteristics of Limited Sight Distance Sites

Certain factors are generally accepted as generating higher accident rates. These factors include the presence of intersections, narrow shoulders or medians, and violations of driver expectancy such as a left rather than a right exit. It has been suggested that the presence of limited sight distance curves should be added to the list of items that generate higher accident rates. As explained in the previous section, a case study approach was chosen for this research because previous studies suggested that relating accidents to limited sight distance curves was not possible with available data bases.

The findings from the case studies are presented in a later section; however, a general overview of whether the study sites were representative of all rural sites was desirable before evaluating individual accident reports. While this effort was not a comprehensive comparative study (where sites with limited sight distance curves were compared to similar sites without limited sight distance curves), comparisons of the study sites with other rural sites were of general interest and thought to be informative. If any differences were identified, additional investigation could be focused in those areas.

The selected study sites were carefully chosen so that any safety effects attributable to limited sight distance curves would be noticeable; i.e., study sites were selected to minimize the effects of as many known accident causal factors as possible. This strategy allowed investigation of the hypothesis that differences in accident rates between rural sites with limited sight distance curves and all rural sites are attributable to the limited sight distance curves.

Note that caution should be exercised when drawing conclusions from comparisons of accident rates at the limited sight distance study sites with published accidents rates from other sources. Different accident reporting practices and roadway geometry may suggest a difference that does not exist or no difference when one does exist. Limited comparisons, however, provide an appreciation of how the selected study sites compare with all rural sites.

Interstate Sites. Due to the long distances between limited sight distance curves on Washington interstates, each limited stopping sight distance curve was identified as a unique accident site. The accident site length was 0.25 miles on both sides of the limited sight distance curve VPI unless the limits for successive vertical curves overlapped; in which case, the overlapping vertical curves were combined into a single accident site. An in-depth analysis of contributing factors to the interstate accidents was not performed because of the large number (826). To evaluate characteristics of the interstate accidents, "one-line accident summaries" were requested from Washington state. These summaries contained computer codes of several of the accident characteristics including pavement condition and type of accident. Table F-9 summarizes the accident characteristics and rates for the interstate sites in Washington.

TABLE F-9. Washington Interstate Accident Summary.

Route	Number of Accidents						Accident Rates	
	Per Yr	Total	Truck*	Snow	Ice	Wet	Acc/MVM	Acc/Yr/Miles
WA-A01	9.6	48	13	19	13	2	2.74	19.20
	8.6	43	3	17	7	9	2.44	17.20
	13.2	66	9	30	13	12	2.40	16.92
	4.6	23	4	5	6	2	1.31	9.20
WA-A02	4.6	23	3	4	4	2	1.44	9.20
	8.6	43	6	10	13	4	2.53	17.20
	9.0	45	11	6	24	6	2.41	18.00
WA-A03	1.8	9	1	3	1	1	0.35	3.60
	12.0	60	5	2	4	21	1.29	16.00
	14.8	74	2	5	4	34	2.39	29.60
	37.0	185	17	20	10	34	1.99	28.68
	31.2	156	13	5	7	49	3.13	62.40
	2.0	10	2	1	1	2	0.28	4.00
	7.0	35	3	1	16	7	1.13	14.00
WA-A04	1.2	6	1	1	1	0	0.35	2.86
Total/Average	11.0	826	93	129	124	185	1.81	18.90

**Truck = Large Truck Related Accidents*

The majority of the Washington interstate accidents (545 of 826) occurred during the day. The most frequent factor contributing to accidents were snow, ice, or water on the pavement. For example, more than 70 percent of the accidents on five of the seven study sites on WA-A01 and WA-A02, were snow/ice/wet pavement related. Close to one-third of the accidents on WA-A01 and WA-A02 involved multiple vehicles while another one-third of the accidents involved single vehicles striking a roadside appurtenance. The site on WA-A04 was the northbound ramp at a freeway interchange. Two of the six accidents on the ramp in the five-year period involved snow or ice on the pavement.

Although WA-A03 was classified as a rural interstate, it carried AADTs between 28,300 and 54,700 in the sections studied. The percentage of snow/ice/wet pavement accidents for the seven sites on WA-A03 ranged from 35 to 69 percent, not as high as on the WA-A01 and WA-A02 sites, but still significant numbers. Approximately 70 percent of the accidents on WA-A03 were multi-vehicle accidents, probably because of the high traffic volumes. The section with the highest accident rate (both Acc/MVM and Acc/Yr/Mi) was the half-mile segment on WA-A03 with the highest AADT (54,700) of any of the freeway sections examined. More than 75 percent of the accidents at that site (118 of 156) were ramp related.

Multilane Sites. The multilane sites had lower accident rates than the interstate sites but higher accident rates than the two-lane sites. As shown in Table F-10, the two multilane sites had an average of 0.87 accidents per million vehicle miles and 2.14 accidents per year per mile. The accidents per million vehicle miles for the two sites were 0.57 and 1.16, with the primary difference being the number of wet pavement-related accidents at one of the sites.

The percentage of intersection/driveway-related accidents, large truck related accidents, and wet pavement related accidents were 16, 8, and 20 percent, respectively. As noted, all of the wet pavement related accidents occurred at one of the two sites. Approximately 36 percent of the accidents at the two sites occurred at night.

Two-Lane With Shoulders Sites. The two-lane with shoulder sites had slightly higher accident rates than the multilane sites, but lower accident rates than the two-lane without shoulder sites. As shown in Table F-11, the seven two-lane with shoulder sites had an average of 1.03 accidents per million vehicle miles and 1.64 accidents per year per mile. The accidents per million vehicle miles ranged from 0.48 to 1.73, with the higher rates associated with the sites having the highest number of intersection and wet pavement related accidents.

The percentage of intersection/driveway-related accidents, large truck related accidents, and wet pavement related accidents were 25, 2.5, and 10 percent respectively. Note that most of the intersection/driveway related accidents and all of the wet pavement related accidents occurred at two of the seven sites. Approximately 54 percent of the accidents on the two-lane with shoulders roadways occurred at night (44 of the 81 accidents).

Two-Lane Without Shoulders Sites. The two-lane without shoulder sites had higher accident rates than the multilane and two-lane with shoulder sites. As shown in Table F-12, the 24 two-lane without shoulder sites had an average of 1.66 accidents per million vehicle miles and 1.71 accidents per year per mile. With one exception (a site with an accident rate of 4.35 accidents per million vehicle miles), the accidents per million vehicle miles ranged from 0.39 to 2.50. The number of accidents per year per mile ranged between 0.12 and 4.04.

The percentage of intersection/driveway related accidents, large truck related accidents, and wet pavement related accidents were 22, 8, and 14, respectively. Note that eight of the sites had no intersection or driveway related accidents and that four of the sites had more than 50 percent of the intersection related accidents. Note also that 10 of the 26 truck accidents occurred at the site that had the highest number of intersection related accidents. Approximately 47 percent of the accidents on the two-lane without shoulder roadways occurred at night.

TABLE F-10. Multilane Accident Summary.

Route	Number of Accidents*					Accident Rates		% Appr Acci
	Per Yr	Total	Appr	Truck	Snow/ Ice/Wet	Acc/ MVM	Acc/Yr/Mi les	
TX-A01	5.67	17	3	1	5	1.16	6.30	18
TX-A02	2.67	8	1	1	0	0.57	0.89	13
Average/Total	4.17	25	4	2	5	0.87	2.14	16

Total column represents three years of data.
**Appr = Approach-related (intersection or driveway) accidents*
Truck = Large truck-related accidents

TABLE F-11. Two-Lane with Shoulders Accident Summary.

Route	Number of Accidents*					Accident Rates		% Appr Acci
	Per Yr	Total	Appr	Snow/ Ice/Wet	Truck	Acc/ MVM	Acc/Yr/Mi	
IL-A01	9.33	28	5	5	2	1.73	2.12	18
TX-A03	0.67	2	0	0	0	0.61	0.74	0
TX-A04	7.67	23	8	3	0	1.54	2.56	35
TX-A05	3.00	9	2	0	0	0.48	1.30	22
WA-A05	2.10	11	3	0	0	0.67	0.79	27
WA-A06	0.60	3	0	0	0	0.80	0.86	0
WA-A07	1.00	5	2	0	0	0.98	1.25	40
Average/Total	**3.48**	**81**	**20**	**8**	**2**	**1.03**	**1.64**	**20**

Total column represents three years of data for Illinois and Texas sites and five years of data for Washington sites.
**Appr = Approach-related (intersection or driveway) accidents*
Truck = Large truck-related accidents

TABLE F-12. Two-Lane without Shoulders Accident Summary.

Route	Number of Accidents*					Accident Rates		% Appr Acc
	Per Year	Total	Appr	Truck	Snow/ Ice/Wet	Acc/ MVM	Acc/Yr/M iles	
IL-A02	6.00	18	4	1	2	1.82	3.53	22
IL-A03	21.00	63	14	10	9	2.31	4.04	22
IL-A04	6.00	18	3	2	2	2.50	3.53	17
IL-A05	2.00	6	0	0	0	1.64	1.33	0
IL-A06	1.67	5	0	0	1	0.98	0.54	0
TX-A06	6.00	18	9	1	2	1.58	1.58	50
TX-A07	1.00	3	1	0	0	0.39	0.45	33
TX-A08	6.33	19	11	0	0	1.29	2.35	58
TX-A09	3.00	9	2	0	2	2.00	0.75	22
TX-A10	5.33	16	6	2	1	4.35	2.22	38
TX-A11	3.33	10	2	0	2	2.20	2.78	20
TX-A12	2.67	8	3	0	0	1.71	1.21	38
TX-A13	2.00	6	3	0	2	2.22	0.71	50
TX-A14	0.67	2	0	0	0	0.89	0.29	0
WA-A08	2.30	14	1	0	2	1.31	1.87	7
WA-A09	1.20	6	0	0	2	1.40	1.33	0
WA-A10	1.80	9	0	1	2	1.10	1.12	0
WA-A11	3.60	18	0	2	4	2.47	1.89	0
WA-A12	3.00	15	1	0	2	0.92	2.00	7
WA-A13	3.60	18	2	4	2	1.56	1.00	11
WA-A14	2.40	12	0	1	5	2.22	1.20	0
WA-A15	3.40	17	4	0	2	1.32	2.27	24
WA-A16	0.20	1	0	0	0	0.41	0.12	0
WA-A17	4.40	22	8	2	4	1.79	2.93	36
Average/Total	**3.87**	**333**	**74**	**26**	**48**	**1.66**	**1.71**	**19**

Total column represents three years of data for Illinois and Texas sites and five years of data for Washington sites.
**Appr = Approach-related (intersection or driveway) accidents*
Truck = Large truck-related accidents

Overall Accident Rates. In a study that examined whether the HSIS data base could be used for accident prediction, accident rates were computed for sections of urban and rural freeways and two-lane highways in two of the states in the HSIS data base (*12*). The other three HSIS states could not be used because their data bases did not have the required accident, geometric, and traffic variables to subdivide the highways into the eight roadway types of interest, or the data were not available to assign accidents to the appropriate sections of highway. Highway sections were defined as not including intersections or interchanges; i.e., the same definition used to select the limited stopping sight distance sites for this study.

The HSIS study's objective was to combine accident data from the two states to provide a single estimate; however, the authors noted that factors such as weather conditions, terrain conditions, roadway types, and accident rates, should be the same before different accident data bases could be merged. They suggested that if the accident rates for similar highway types were not significantly different, the data bases could be combined, but if the accident rates were significantly different, the data bases should not be combined. Their results indicated that the accident rates for the rural two-lane highways in the two states were significantly different and could not be combined; however, the reported accident rates provide an estimate of the range in accident rates for rural two-lane highways.

The two states had total accident rates of 1.07 and 1.86 accidents per million vehicle-miles of travel per year, respectively (*12*). For the limited sight distance sites, the average accident rate for the two-lane without shoulders sites in Washington, Illinois and Texas were 1.45, 1.85, and 1.85 accidents per million vehicle miles, respectively, all of which are between the accident rates for the two HSIS states. The average accident rate for the two-lane with shoulder sites was 0.97 accidents per million vehicle miles, which is slightly below the accident rate for the HSIS states. Thus, it was concluded that the accident rates for the limited stopping sight distance sites are similar to the accident rates for rural two-lane highways in the two HSIS states.

Older/Younger Driver Accidents. One question of interest was whether roadways with limited stopping sight distance vertical curves create significant safety problems for older drivers and/or inexperienced drivers. A comparison of the percentage of the accidents on roadways with limited stopping sight distance vertical curves involving older and younger drivers to the percentage of all accidents involving older and younger drivers should indicate whether limited stopping sight distance presents a significant problem for either type of driver.

Of the 609 drivers involved in accidents at the selected study sites, 91 of them (14.9 percent) were 55 years of age or older. The National Safety Council reports that 14.8 percent of the drivers involved in accidents are 55 years of age or older (*13*). These similar percentages suggest that roadways with limited stopping sight distance vertical curves are not a significant problem for the older driver. Examination of the number of younger drivers involved in accidents at the selected study sites produced similar findings. Approximately, 16.1 percent (98 drivers) of the drivers at the study sites were 20 years of age or younger while the National Safety Council reports that 17.2 percent of the drivers involved in accidents are 20 years of age or younger (*13*). Thus, the data suggests that neither older nor younger drivers are over represented in accidents at limited stopping sight distance vertical curves.

Large Truck Accidents. A second question of interest was whether limited stopping sight distance vertical curves create safety problems for large trucks because of their generally poorer braking performance. In other words, even though the truck's higher driver eye height offsets some of their additional braking requirements, there is concern that the current SSD model is not adequate for large trucks. If the current model really does create safety problems for large trucks, the percentage of large truck accidents on roadways with limited stopping sight distance vertical curves should be greater than the percentage of large truck accidents on all roadways.

In 1992, 3.4 percent of vehicles involved in all accidents (i.e., urban and rural, etc.) were medium or heavy trucks (*14*). Medium/heavy trucks are defined as truck tractors with or without the semi-trailer. For the limited stopping sight distance sites in this study, 2 large trucks were involved in accidents on multilane roadways, 2 large trucks were involved in accidents on two-lane with shoulder roadways, and 26 large trucks were involved in accidents on two-lane without shoulder roadways (see Tables F-14, F-15, and F-16).

These 30 large truck accidents represent 4.9 percent (30 of the 609 vehicles) of the accidents investigated; however, 10 of the 30 accidents involving large trucks occurred at a single site. This site also experienced the largest number of intersection related accidents. The percentage of accidents involving large trucks at the 33 study sites is 4.9 percent. If the site with 10 truck accidents is removed from the sample, the percentage of truck related accidents drops to 3.3 percent. These percentages are similar to the percentage of large trucks involved in accidents on all roadway types (3.4 percent) and suggest that large trucks are not over represented in accidents on roadways with limited stopping sight distance vertical curves.

Comparison Between Limited Sight Distance Sites

Several methods of classifying the data (by roadway type, by geographic region, and by design speed) were developed to investigate the variation in accident rates between the different sites. These comparisons were presented graphically, and the results of the analysis indicated very small differences in accidents per million vehicle miles or accidents per year per mile for any of the classification schemes. The following sections discuss the graphical analysis and comparison of accident rates in terms of accidents per million vehicle miles.

Classification by Roadway Type. The sites were first classified according to roadway type (multilane, two-lane with shoulders, two-lane without shoulders) to test for differences by roadway type. It should be noted that for this study *with shoulder* sites were those with paved shoulders six feet in width or greater, while *without shoulder* sites were those with unpaved or narrow (less than six feet in width) paved shoulders. Figure F-9 illustrates the accident rate in terms of accidents per million vehicle miles as a function of limited stopping sight distance roadway type.

The average accident rate was 0.86 accidents per million vehicle miles for the multilane sites, 0.97 accidents per million vehicle miles for the two-lane with shoulder roadways, and 1.68 accidents per million vehicle miles for the two-lane without shoulder roadways. Note, however, that the accident rate at one of the two-lane without shoulder roadways is much higher than at the other 23 sites, and adds greatly to the within roadway variability. Analysis of variance tests at the 95 percent confidence level indicated that there was no difference in mean accident rate for the three types of roadways.

Classification by State. The sites were classified by state to determine if sites with limited stopping sight distance vertical curves in one state had significantly higher or lower accident rates than similar sites in the other two states. It was not possible to test for differences in multilane accident rates by state because the only two multilane sites were both in Texas. The graphical analysis of the accident rates on two-lane with shoulder roadways is not shown because there were only seven sites of this type. Figure F-10 illustrates the accident rate in terms of accidents per million vehicle miles on two-lane without shoulder roadways by state.

The two-lane without shoulder roadways' average accident rate was 1.85 accidents per million vehicle miles for Illinois sites; 1.45 accidents per million vehicle miles for Washington sites; and, 1.85 accidents per million vehicle miles for Texas sites. Analysis of variance tests at the 95 percent confidence level indicated that there was no difference in accident rates on two-lane without shoulder roadways with limited stopping sight distance for the three states in this study.

Classification by Design Speed and Subjective Rating. The sites were then classified by subjective rating and mean design speed of the limited sight distance crest curves within the segment to test for differences in accident rates due to curve environment and design speed. As noted in the *Methodology* section, sites with a *subjective rating of 1* represented an isolated curve and sites with a *subjective rating of 3* represented several closely spaced curves with a consistent design speed. Sites with a *subjective rating of 2* were those sites that could not be classified as a *1* or a *3*; i.e., multiple curves and variable curve geometry within the section.

As shown in Figure F-11, both multilane sites had average design speeds in the 45 to 50 miles per hour category, Thus, it was not possible to make design speed comparisons *within* subjective rating categories; however, it was possible to make comparisons *between* subjective rating categories. The accident rate at the isolated curve site (*subjective rating 1*) was higher than the accident rate at the site with many consistent curves (*subjective rating 3*)—1.16 accidents per million vehicle miles compared to 0.57

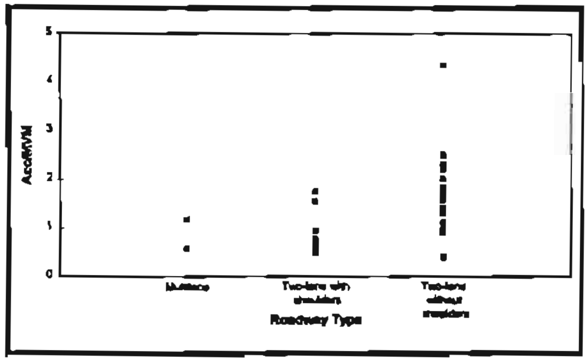

FIGURE F-9. Classification by Roadway Type-Accidents per Million Vehicle Miles.

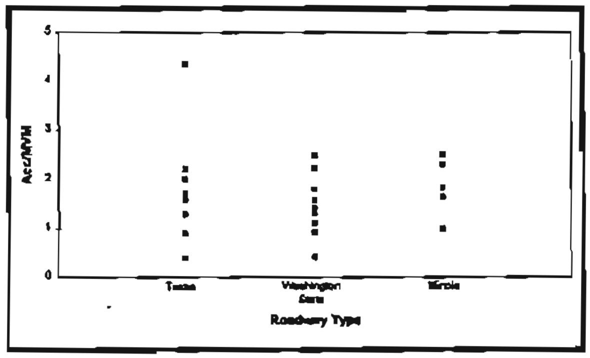

FIGURE F-10. Classification of Two-Lane Roadways without Shoulders by State-Accidents per Million Vehicle Miles.

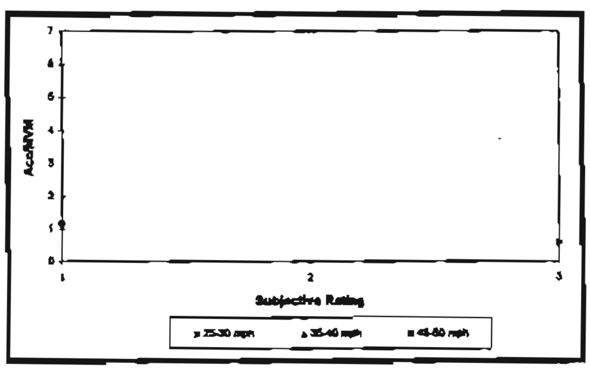

Figure F-11. Classification of Multilane Roadways by Mean Design Speed-Accidents per Million Vehicle Miles.

accidents per million vehicle miles. Unfortunately, strong conclusions regarding this trend are not possible with only two sites.

Figure F-12 illustrates the accident rates on limited stopping sight distance two-lane roadways with shoulders. As shown, all of the sites in *subjective rating 1* and *subjective rating 3* had a mean design speed of 45 to 50 mph. Thus, as with the multilane sites, it was not possible to make comparisons *within* the subjective rating category; however, a comparison could be made *between* the subjective rating categories. Note that accident rates at the isolated curve sites (*subjective rating 1*) were noticeably lower than the accident rates at the sites with many consistent curves (*subjective rating 3*)—0.79 accidents per million vehicle miles compared to 1.73 vehicle miles. This result suggests that accident rates may increase with increases in the number of limited sight distance vertical curves.

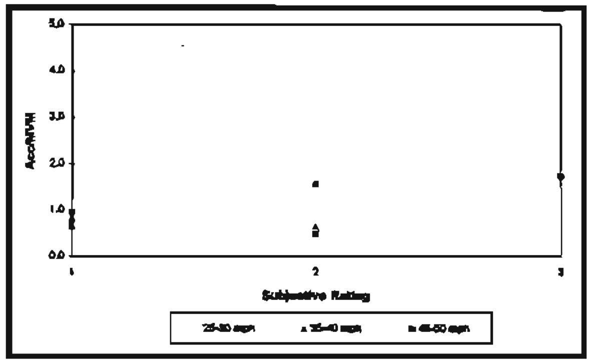

Figure F-12. Classification of Two-Lane Roadways with Shoulders by Mean Design Speed-Accidents per Million Vehicle Miles.

Figure F-13 illustrates the accident rates on limited stopping sight distance two-lane without shoulder roadways as a function of subjective rating and design speed. Note that there are multiple design speeds in each of the subjective rating categories. With the exception of two sites, the sites with the highest accident rates for *subjective ratings 1 and 3*, this comparison suggests that sites with *higher* accident rates correspond to sites with a *lower* design speed; i.e., higher accident rates were associated with shorter stopping sight distances. As mentioned, it was difficult to identify trends in the *subjective rating 2* category due to the wide range of geometry it represents.

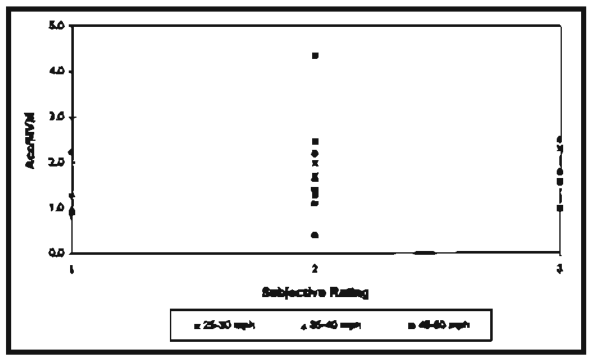

Figure F-13. Classification of Two-Lane Roadways without Shoulders by Mean Design Speed—Accidents per Million Vehicle Miles.

The two two-lane without shoulder sites that do not fit this trend were WA-A14 and IL-A04. The first site, WA-A14, was an isolated vertical curve (*subjective rating of 1*) with a design speed of 45 to 50 mph. The accident rate at this site was 2.22 accidents per million vehicle miles which is higher than the rate at a similar site with a design speed of 35 to 40 mph; however, the increased accident rate at WA-A14 can be explained by looking at the contributing factors to the accidents at this site.

Five of the 12 accidents (42 percent) at this site had snow/ice on the pavement as a contributing factor. This percentage of snow/ice related accidents is the highest for any of the 27 two-lane without shoulders roadway sites and suggests that the snow/ice related accidents are inflating the accident rate at WA-A14. The accident rate at WA-A14 without the five snow/ice related accidents is 1.23 accidents per million vehicle miles—about the same as the other site with a design speed of 45 to 50 mph and less than the site with the design speed of 35 to 40 mph.

The second site that appeared to have an overly high accident rate, IL-A04, contained several vertical curves (*subjective rating of 2*) with a mean design speed 45 to 50 mph. After reanalyzing the K-values for the five vertical curves at this site, it was discovered that one of the vertical curves had a K-value of 143, and the other four vertical curves had K-values between 58 and 79. Including the 143 inflated the average K-value of the other curves from 68.6 to 83.5. More importantly, including this K-value in the average raised the mean design speed category from 35 to 40 mph to 45 to 50 mph. In addition, this site's accident rate is similar to the other sites in the 35 to 40 mph design speed category which suggests that it should be reclassified.

Figure F-14 illustrates the accidents per year per mile on limited stopping sight distance two-lane without shoulder roadways as a function of subjective rating and design speed. It is interesting to note that three of the four sites with several consistent curves (*subjective rating 3*) and a mean design speed of 45 to 50 mph had accident rates between 0.54 and 1.58 accidents per year per mile, and the sites with the same subjective rating and a mean design speed of 35 to 40 mph had accident rates between 2.79 and 4.04 accidents per year per mile. In other words, higher accident rates were generally associated with lower design speeds (shorter stopping sight distances).

The one 45 to 50 mph design speed site that did not follow this trend was IL-A04. As noted earlier, a single vertical curve with a high K-value raised the average K-value for the site from the 35 to 40 mph design speed category to the 45 to 50 mph design speed category. If that K-value is removed from the average, the site falls into the lower design speed category and the accident rate is in the range for the lower design speed category. A pooled t-test (without the data from site IL-A04) indicated that there was a significant difference in accident rates between the 35 to 40 mph design speed sites and the 45 to 50 mph design speed sites at the 95 percent confidence level. Higher accident rates were associated with 35 to 40 mph design speed for two-lane without shoulder roadways.

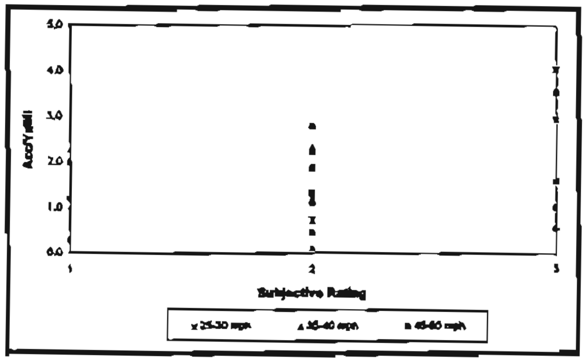

Figure F-14. Classification of Two-Lane Roadways without Shoulders by Mean Design Speed—Accidents per Year per Mile.

Accidents per Approach. Vehicles entering and exiting the traffic stream from driveways and intersections represent potential conflicts and are a major contributing factor to accidents on rural highways. As mentioned, sites with major driveways and intersections were eliminated from consideration during the site selection procedure; however, there were still minor driveways and intersections within most of the selected sites. The number of driveway approaches per mile was determined by reviewing the field notes made during the site selection trips. The actual numbers for Texas, Washington, and Illinois sites are illustrated in Tables F-4, F-6, and F-7, respectively.

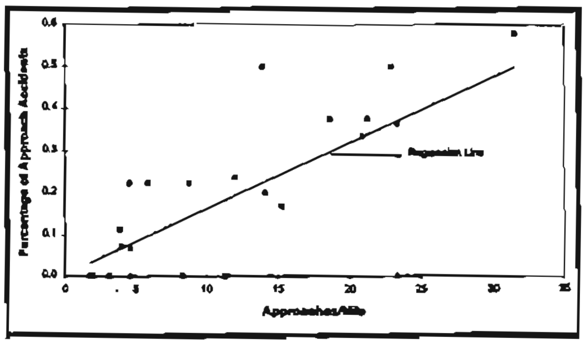

Figure F-15. Approach Accidents versus Number of Approaches per Mile.

The number of driveway approach related accidents was determined by reviewing the accident narratives. Approach related accidents were identified as any accident that stated that a vehicle was entering or leaving a driveway or intersection. It was not possible to draw conclusions regarding the relationship between the percentage of approach related accidents and number of driveway approaches per mile in the multilane and two-lane with shoulder roadway categories due to the limited number of sites in those categories.

Figure F-15 illustrates the relationship between the percentage of approach-related accidents and the number of driveway approaches per mile for the two-lane without shoulder roadway study sites. This figure suggests a relationship between these two variables, and a linear regression analysis was performed to determine the correlation, or strength of the relationship. From the analysis of the two-lane without shoulders roadway data, it was determined that the correlation between the two variables (r^2) was 0.54. Thus, approaches per mile is a moderately good predictor of the percentage of approach-related accidents.

Examination of Individual Accidents

The following sections discuss the most frequent contributing factors to accidents on limited stopping sight distance roadways and the three contributing factors most logically associated with limited stopping sight distance accidents: limited stopping sight distance (LSSD), headlight sight distance, and object in the roadway.

Contributing Factors. Each individual accident report was reviewed to determine contributing factors to the accident. While the accident narrative provided the greatest amount of information regarding the cause of the accident, information on the specific elements or sequence of events which contributed to individual accidents was supplemented from several sources. For example, the roadway geometry available from plan and profile sheets provided information on the horizontal and vertical curvature near the accident site, and data on the accident report in addition to the narrative box provided information such as light and pavement condition. Approximately half of the accidents had more than one contributing factor.

Reviewing the entire list of contributing factors to accidents on roadways with limited stopping sight distance gives an overview of characteristics that appear over represented in the data base. Tables F-13, F-14, and F-15 list the most frequent contributing factors for the multilane sites, two-lane with shoulder sites, and two-lane without shoulder sites, respectively. Of the 499 contributing factors to accidents on the two-lane without shoulder sites, 81 were in the object-related category; i.e., 81 of the 333 accidents (24 percent) involved objects (such as a tree branch or another vehicle)in the roadway. The most frequent contributing factor to accidents on the two-lane with shoulder sites also was an object in the roadway; i.e., almost 40 percent of the accidents on the two-lane with shoulder sites involved objects in the roadway. Eight percent of the accidents on multilane highways involved objects in the roadway.

TABLE F-13. Multilane Contributing Factors.

State Route	Contributing Factors*					
	Object	HLSD	Speed	Wet	Imp	Other
TX-A01	2	1	8	5	4	4
TX-A02	0	0	3	0	1	10
TOTAL	**2**	**1**	**11**	**5**	**5**	**14**

**Object = Object on Road; HLSD = Headlight Sight Distance*
Speed = Speed too Fast for Conditions; Wet = Water on Pavement
Imp = Driver was Impaired; Other = All Remaining Contributing Factors

TABLE F-14. Two-Lane with Shoulders Contributing Factors.

State Route	Contributing Factors*					
	Object	HLSD	Speed	Wet	Neg	Other
IL-A01	15	0	5	4	1	11
TX-A03	2	2	0	0	0	0
TX-A04	7	4	6	2	0	19
TX-A05	2	1	0	0	0	7
WA-A05	3	1	2	0	2	6
WA-A06	1	1	0	0	1	1
WA-A07	2	1	0	0	3	3
TOTAL	**32**	**10**	**13**	**6**	**7**	**47**

**Object = Object on Road*
HLSD = Headlight Sight Distance
Speed = Speed too Fast for Conditions
Wet = Water on Pavement
Neg = Driver was Negligent
Other = All Remaining Contributing Factors

TABLE F-15. Two-Lane without Shoulders Contributing Factors.

State Route	Contributing Factors[1]						
	Object	HLSD	Speed	Icy	Imp	Neg	Other
IL-A02	8	6	4	1	1	1	15
IL-A03	17	9	4	5	5	3	43
IL-A04	11	5	0	0	0	0	2
IL-A05	6	1	0	1	0	0	0
IL-A06	4	4	2	2	0	0	7
TX-A06	2	0	4	2	2	2	14
TX-A07	0	0	0	0	0	0	3
TX-A08	1	0	3	0	5	0	14
TX-A09	1	0	6	0	0	0	13
TX-A10	1	0	3	0	1	1	14
TX-A11	4	2	5	0	0	1	7
TX-A12	1	0	2	0	1	0	8
TX-A13	1	0	2	1	1	0	5
TX-A14	0	0	1	0	1	0	0
WA-A08	4	4	2	2	0	4	9
WA-A09	0	0	3	2	1	1	5
WA-A10	3	1	3	2	0	1	6
WA-A11	2	1	2	4	3	1	16
WA-A12	3	1	1	2	3	4	4
WA-A13	2	1	2	1	1	3	16
WA-A14	3	0	3	5	0	1	12
WA-A15	5	3	3	2	1	2	9
WA-A16	0	0	0	0	0	0	1
WA-A17	2	0	2	1	2	4	14
TOTAL	81	38	57	33	28	29	233

[1]*Object = Object on Road*
HLSD = Headlight Sight Distance
Speed = Speed too Fast for Conditions
Icy = Snow or ice on Pavement
Imp = Driver was Impaired
Neg = Driver was Negligent
Other = All Remaining Contributing Factors

A subset of the object-related accidents were those accidents in which the object was struck at night. This contributing factor category was called *headlight sight distance* and represented those accidents where stopping sight distance probably was limited more by the vehicle's headlights than by the roadway's geometry. Slightly more than 10 percent of the accidents at the two-lane without shoulder sites, 12 percent of the accidents at the two-lane with shoulder sites, and 4 percent of the accidents at the multilane sites were accidents where an object was struck at night.

The second most frequent contributing factor to accidents on all three types of roadways was *speed too fast for conditions.* This factor contributed to approximately 17 percent of the two-lane roadway accidents and 44 percent of the multilane roadway accidents at the study sites. Other frequent contributing factors to accidents at the study sites were wet or icy pavements—10 percent of the two-lane roadway accidents and 20 percent of the multilane roadway accidents; driver impairment—8 percent of the two-lane without shoulder roadway accidents and 20 percent of the multilane roadway accidents; and driver negligence—8 percent of the two-lane roadway accidents.

Limited Stopping Sight Distance. Judgement was utilized when determining if the accident occurred on or near a crest curve because of the previously mentioned limitations in accurately determining an accident's exact location. Because of the desire to determine how or if limited stopping sight distance contributes to accidents, any accident where limited stopping sight distance *could* have been a contributing factor was identified as a *limited stopping sight distance* (LSSD) related accident. Limited stopping sight distance was identified as a contributing factor if the accident was believed to have occurred on or near a limited sight distance curve, the accident occurred during daylight conditions, and there was an object in the roadway. In other words, the crest of the vertical curve limited the driver's view of an object in the roadway.

None of the accident narratives specifically stated that the accident was caused by limited sight distance; however, several accident narratives with stopping sight distance as a contributing factor stated that the hillcrest blocked the driver's view. For example, the narrative for one of the accidents on WA-A14 stated: "Hit deer. Did not see it in time to slow down or avoid it." Limited stopping sight distance was identified as a possible contributing factor to this accident because it occurred near a limited sight distance vertical curve.

Other accident narratives provided more clues into the possible involvement or contribution of the crest vertical curve. For example, the narrative for an accident where a vehicle stuck another vehicle that had been involved in a previous accident stated that "Unit 1...drove up a small hill..." Because this accident occurred at night (4:49 a.m.) and at a crest vertical curve, its contributing factors were identified as *headlight sight distance* and *object in the road*. Another example of sight distance as a contributing factor to an accident was the following narrative: "Unit 1 ... was stopped for a school bus unloading when unit 2...came over a hill and was unable to stop veered to the left of roadway striking unit 1."

Even though the sites were selected to maximize the possibility of stopping sight distance accidents, limited stopping sight distance was involved in only a small portion of the accidents at the study sites. Only 14 of the 439 accidents investigated (3 percent) had limited stopping sight distance as a possible contributing factor to the accident. By roadway type, 3.0 percent (10 of 333 accidents) of the accidents on the two-lane without shoulder sites, 4.9 percent (4 of 81 accidents) of the accidents on two-lane with shoulder sites, and none of the accidents on the multilane sites had limited stopping sight distance as a possible contributing factor to the accident.

Table F-16 summarizes the 14 accidents with limited stopping sight distance as a possible contributing factor to the accident. Note that the majority of the accidents (9 of 14) involved striking another vehicle. The non-vehicle objects struck were deer (two), deep water, a dog, and an unknown object. Only one of the accidents with sight distance as a possible contributing factor involved a large truck. A farm truck pulling a small trailer was traveling on a parallel, private access road to the two-lane roadway. The trailer came loose and entered the two-lane highway. A gravel dump truck struck the trailer head-on, left the roadway, and overturned.

Four of the 24 drivers (17 percent) involved in accidents with limited stopping sight distance as a possible contributing factor were 55 years of age or older. Another four drivers (17 percent) were 20 years in age or younger. These percentages are near those reported by the National Safety Council (*13*) which reports that 14.8 percent of the drivers involved in all accidents are 55 years or older and 17.2 percent are 20 years or younger. These data suggest that older or younger drivers are not over represented in limited stopping sight distance related accidents.

TABLE F-16. Accidents with Sight Distance as a Contributing Factor.

Route	Class[a]	Object Struck	Age of Driver[b]	K-value	Subject Rating
IL-A02	w/o	Vehicle	B	50	3
IL-A02	w/o	Vehicle	B	125	3
IL-A03	w/o	Vehicle	B	30	3
IL-A03	w/o	Vehicle	B	65	3
IL-A01	with	Deer	Older	122	3
TX-A04	with	Vehicle	Younger	92	2
TX-A04	with	Vehicle	B	92	2
TX-A09	w/o	Vehicle	B	41	2
TX-A12	w/o	Vehicle	Younger	43	2
WA-A07	w/o	Dog	B	105	1
WA-A13	w/o	Deep Water	Younger	75	3
WA-A13	w/o	Vehicle	B	75	3
WA-A14	w/o	Unknown	Older	122	1
WA-A14	w/o	Deer	Younger	122	1

[a] *All accidents listed in above table occurred on two-lane highways. When 6 feet or more of paved shoulder is present, the roadway is classified as having shoulders (with); when less than 6 feet of paved shoulders is present, the 2-lane roadway is without shoulders (w/o).*

[b] *The age of one of the involved drivers was 20 years old or younger (younger), 55 years old or older (older), or the accident did not involve a younger or older driver (B).*

Headlight Sight Distance. Accidents with *headlight sight distance* as a contributing factor were defined as those accidents in which a driver struck an object in the roadway at night. These accidents could occur on vertical curves, horizontal curves, or tangent sections of roadway. Table F-17 lists the 19 accidents that occurred at night, near a vertical curve, and involved striking an object in the roadway. The table is divided into two sections—those accidents where limited stopping sight distance could have been a contributing factor and those accidents where limited sight distance was not believed to be a contributing factor. An example of the latter situation is a deer running into a vehicle rather than a vehicle striking a deer. Interestingly, 15 of the 19 *headlight sight distance* accidents that occurred near crest vertical curves involved a deer in the roadway.

Twenty-seven additional accidents beyond those listed in Table F-17 had headlight sight distance as a contributing factor to the accident. The characteristics of these accidents were that they occurred at night (i.e., headlights were in use), they involved striking an object, and they occurred on a tangent section of roadway (i.e., stopping sight distance was not limited by a vertical curve). Of these 27 headlight related accidents, a deer was struck in 18 accidents, a bull or cow was struck in six accidents, another vehicle was struck in two accidents, and a tree was struck in one accident.

TABLE F-17. Accidents with Headlight Sight Distance as a Contributing Factor.

State Route	Class[a]	Object Struck	Age of Driver[b]	K-Value of Curve	Subjective Rating
Accidents where sight distance could have been a contributing factor					
IL-A03	w/o	Deer	B	40	3
TX-A05	with	Vehicle	B	93	2
WA-A08	w/o	Deer	Older	71	2
WA-A08	w/o	Deer	B	71	2
Accidents that occurred on a vertical curve where sight distance is not believed to be a contributing factor					
IL-A02	w/o	Deer	B	50	3
IL-A02	w/o	Deer	B	50	3
IL-A02	w/o	Deer	B	50	3
IL-A03	w/o	Deer	B	136	3
IL-A03	w/o	Deer	B	95	3
IL-A03	w/o	Deer	B	95	3
IL-A03	w/o	Deer	Older	95	3
IL-A04	w/o	Deer	B	58	3
IL-A04	w/o	Deer	B	143	3
IL-A05	w/o	Deer	B	90	2
TX-A04	with	Vehicle	B	104	2
WA-A08	w/o	Deer	B	71	2
WA-A07	with	Deer	B	105	1
WA-A11	w/o	Pony	Older	84	2
WA-A13	w/o	Bike	B	109	3

[a] *All accidents listed in above table occurred on two-lane highways. When 6 feet or more of paved shoulder is present, the roadway is classified as having shoulders (with); when less than 6 feet of paved shoulders is present, the 2-lane roadway is without shoulders (w/o).*

[b] *The age of one of the involved drivers was 20 years old or younger (younger), 55 years old or older (older), or the accident did not involve a younger or older driver (B).*

Object in Roadway. The contributing factor, *object-related*, was used for all accidents in which a driver struck an object in the roadway. The same contributing factor designation was used for both day and night conditions. Somewhat surprisingly, the percentage of *object-related* accidents was not consistent between the three states. Texas and Washington had similar percentages (16 percent in Texas and 20 percent in Washington), but the *object-related* accident percentage was much higher in Illinois. Over 44 percent of the accidents in Illinois involved striking an object in the roadway. The cause for this disparity is a significantly larger number of deer accidents in Illinois (over 6,000 in 1990).

As shown in Table F-18, deer were the most common type of object struck (83 of 115 objects). In addition, most of the objects struck are large objects (104 of 115). This data suggests that even on limited sight distance roadways, the six-inch object does not appear to be the critical situation in terms of contributing factors to accidents. In other words, everyone knows that small unexpected objects in the roadway exist, but there is no evidence that they cause accidents. The most probable reason for the lack of accidents is that drivers are able to avoid or run over the smaller size objects without damaging their vehicles.

As noted, deer-related accidents were a major cause of safety problems on the rural two-lane highways in this study. Deer also are a significant safety problem in a number of other states, and represent a problem that is likely to get worse before getting better. According to recent estimates, the number of white tail deer in the lower 48 states (approximately 25 million) has almost doubled in the past decade and is expected to continue increasing in the future (*15*). The increase in numbers and their behavior around highways may explain why deer are involved in so many accidents on rural highways. Deer are attracted to highways, partly because of salt leeching into the surrounding soil, and partly because of forage planted in the median and along the roadside. Additionally, deer cross roadways to move from open feeding areas to protected bedding areas in regular cycles, sometimes several times a day.

The problem for motorists is that deer react to cars in seemingly illogical ways, especially at night. For example, it appears that deer often wait until a car is quite close and then run out in front of the vehicle just in time to get hit; however, this seemingly illogical behavior to drivers is really quite logical for a deer. Deer freeze as a standard response to an approaching threat, especially under bright lights. Essentially, the bright lights of the approaching vehicle negate their extremely effective night vision. Thus, they may not recognize that a car is approaching, or they may think that they are hidden and cannot be seen. The deer then waits until it is time to escape and their biological reaction is to get in front or ahead of whatever is pursuing them—which, of course, they cannot (*15*).

TABLE F-18. Objects Struck.

Object	Frequency
Deer	83
Vehicle	11
Cow	6
Bull	2
Tree	2
Dog	2
Horse	2
Other	3
Unknown	4
Total	115

SUMMARY

The objective of this study was to determine if available stopping sight distance was a contributing factor in accidents on roadway segments containing limited sight distance crest vertical curves. This objective was accomplished by reviewing the literature and narratives of accidents that occurred on selected multilane and two-lane roadways with limited sight distance crest vertical curves. The findings and recommendations regarding safety effects of stopping sight distance are documented in the following sections.

Findings

Contributing factors and characteristics of the 439 accidents which occurred on the multilane and two-lane roadways with limited stopping sight distance were analyzed. As a result of this analysis, the following conclusions were drawn.

- The accident rates on the 31 rural two-lane highways with limited stopping sight distance in the study's data base are similar to the accident rates on all two-lane rural highways in two large states. Thus, limited stopping sight distance did not appear to cause a safety problem for the sites in this study.

- Approximately four percent (18 of 439) of the accident narratives reviewed had limited stopping sight distance as a possible contributing factor to the accident. Thus, even on limited sight distance roadways, limited stopping sight distance does not appear to be a major safety problem.

- All of the accidents with limited sight distance as a possible contributing factor occurred on crest vertical curves with minimum stopping sight distances of 400 feet or less (K-values of 125 or less). Additionally, most of the accidents with limited stopping sight distance as a possible contributing factor occurred on vertical curves with minimum stopping sight distances of 360 feet or less (K-values of 100 or less). Current AASHTO policy requires a minimum stopping sight distance of 450 feet (K- value of 150) for a 55 mph design speed. Thus, moderate reductions in minimum stopping sight distance do not appear to cause a safety problem.

- Most of the objects struck (104 of 115) on roadways with limited sight distance crest vertical curves were large objects such as deer, cattle, horses, and other vehicles. Thus, small objects do not appear to cause a safety problem.

- The majority of the accidents with limited stopping sight distance as a contributing factor (10 of 18 accidents) were caused by another vehicle stopped in the roadway to make a turn into a driveway or intersection. Therefore, the placement of driveways and intersections should be carefully considered in the design of new roadways or reconstruction of existing roadways.

- The percentage of accidents involving large trucks in this study was comparable to the percentage of all accidents involving large trucks reported by the National Safety Council. Because large trucks are not over represented in accidents on limited stopping sight distance roadways, limited stopping sight distance does not appear to cause a safety problem for large trucks.

- The percentage of accidents involving younger and older drivers in this study was comparable to the percentage of all accidents involving younger and older drivers reported by the National Safety Council. Because younger and older drivers are not over represented in accidents on limited stopping sight distance roadways, limited stopping sight distance does not appear to cause a safety problem for either inexperienced or elderly drivers.

Recommendations

The following recommendations regarding the safety effects of limited stopping sight distance are made:

- Minimum stopping sight distance requirements should be consistent with the point at which an unacceptable accident rate is noted. Based on the literature and this study, that point appears to be somewhere between 300 and 360 feet for speeds of 55 mph. The exact threshold value is dependent on whether hazards are located within the limited stopping sight distance section.

- Because there are no apparent safety benefits from providing stopping sight distances longer than some threshold value, stopping sight distance requirements for reconstruction projects could be based on safety threshold values rather than driver-vehicle performance considerations.

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APPENDIX G -

OPERATING SPEED STUDIES

INTRODUCTION

Stopping sight distance influences a variety of geometric design values including length and sharpness of horizontal and vertical curvature. To provide longer stopping sight distance values, horizontal curves are flattened (degree of curvature is decreased) and vertical curves are lengthened. When designing crest vertical curves, there is a tradeoff between available sight distance and the cost of excavation or fill. Longer stopping sight distances lead to longer vertical curves that, in turn, increase construction costs. Depending on whether minimum or desirable values are used as the design criteria, the construction cost of a particular roadway can increase dramatically.

One of the parameters within the American Association of State Highways and Transportation Officials (AASHTO) stopping sight distance model (SSD model), the initial speed for determining appropriate stopping sight distance, is reviewed in this appendix. Suggested values for this parameter include the design speed for the facility, a lower average running speed for low volume conditions as suggested by the AASHTO guidelines (*1,2,3*), or other speeds that are different from either of these two values. There have been a number of studies relating operating speed to horizontal curve design, but relatively few studies relating operating speed to vertical curve design. Thus, a study of operating speeds on crest vertical curves is needed to provide guidance in this area.

The objective of the operating speed study was to evaluate the relationship between design and operating speeds at crest vertical curves with limited stopping sight distance. To accomplish this objective, the following tasks were performed:

- Review the literature concerning design and operating speed;
- Develop a study design to measure differences in speeds at control (flat, tangent section) and limited sight distance sections;
- Collect field data for statistical analysis; and
- Determine whether a significant difference exists between speeds on control and limited sight distance sections.
- Determine the relationship between design and operating speeds at crest vertical curves with limited stopping sight distance.

The findings from the *Operating Speed* studies are divided into five sections. The first section includes the introduction, problem statement, research objective, and appendix organization. The second section presents a review of the design speed concept, the history of the SSD model, and how certain parameters have changed over time. The third section includes the study design which describes the procedure for selecting the study sites, as well as the procedures for data collection and analysis. The results of the speed studies are presented in the fourth section. The fifth section presents the conclusions and recommendations from these studies.

LITERATURE REVIEW

It is generally agreed that drivers adjust their speed according to the speed limit, their desired speed, perceived hazards or risks, and the influence of other vehicles. Accordingly, the speeds adopted by individual drivers tend to vary and often may be in excess of the design speed of the roadway (*4*). Previous studies have shown that average operating speeds (50th percentile) often exceed the roadway's design speed at horizontal and vertical curves (*5, 6*) and are relatively constant until volume approaches capacity (*7*). Furthermore, observations have shown that drivers do not slow down during wet weather. These findings contradict the premise that most motorists drive slower than the design speed of the road.

This section presents a review of the design speed concept, describes how this concept has changed over time, discusses the disparities between design and operating speed, and reviews the background of the current stopping sight distance model. Changes in the model from the 1940s to present are also presented.

Design Speed Concept

Horizontal and vertical elements of a highway are designed based on an assumed design speed. The design speed concept was developed in the 1930s as a mechanism for designing rural alignments to permit the majority of drivers to operate uniformly at their desired speed. AASHO recognized that drivers will select a speed that is influenced by the roadway environment rather than an assumed design speed as early as 1938 (*1*). They wrote "A low design speed should not be assumed for a secondary road, however, if the topography is such that vehicle operators probably will travel at high speeds...Drivers do not adjust their speed to the importance of the road but to the physical limitations of curvature, grade, sight distance, smoothness of pavement..."

The design speed concept was developed to address concerns resulting from a discrepancy between speeds for which horizontal curves were designed and the speeds at which drivers negotiated those curves. The basis of the use of a design speed concept was a result of work done by Barnett (*8*). Barnett recommended that, "The assumed design speed of a highway should be the maximum reasonably uniform speed which would be adopted by the faster driving group of vehicle operators, once clear of urban areas." He urged that all features of geometric design be consistent with the chosen design speed. The design speed or "balanced design" concept became a permanent feature of geometric design policy in the United States when it was adopted by AASHO in 1938. AASHO defined design speed as "the maximum approximately uniform speed which probably will be adopted by the faster group of drivers but not, necessarily, by the small percentage of reckless ones."

Adoption of this concept posed the problem of deciding the design speed for a particular set of conditions; i.e., What was the "maximum approximately uniform speed adopted by the faster group of drivers?" To answer that question, the Bureau of Public Roads engineers used data from 260,000 vehicles measured at 40 different locations in 1934, 1935, and 1937. They used these data to plot ratios of the speeds of the fastest drivers to the average speeds of all drivers for various percentiles of total traffic. Based on the resulting curves, they recommended that the design speed of a future highway should be the speed that only 2 to 5 percent of the drivers will exceed after the road is built.

More recent research (*9*) argues that "Design speed is no longer the speed adopted by the faster driving group of vehicle operators, but has become a value used for the correlation of design elements which is also a maximum safe speed." Good (*10*) states "...there seems to have been a change in emphasis from design speed as a speed which might be expected from driver behavior, to a speed which is *safe* from the designers point of view."

Current Use of Design Speed

AASHTO (2,*3*) currently defines design speed as "the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern." A fundamental limitation of the design speed concept is that it only applies to horizontal and vertical curves. Design speed has no practical meaning on tangent sections of roadway, and therefore, provides no basis for establishing maximum tangent lengths to promote consistency and control the maximum operating speeds that can be reached. In fact, AASHTO's encouragement of the use of above minimum values may have a negative effect on the consistency among alignment elements. Use of above minimum values may encourage operating speeds that exceed the design speed of the controlling element.

The 1994 Green Book (*3*) provides general guidance on both the selection and application of design speed. Examples of guidance in the Green Book include the following:

- "The assumed design speed should be a logical one with respect to the topography, the adjacent lane use, and the functional classification of the highway."
- "Except for local streets where speed controls are frequently included intentionally, every effort should be made to use as high a design speed as practicable to attain a desired degree of safety, mobility, and efficiency while under the constraints of environmental quality, economics, aesthetics, and social or political impacts."
- "The design speed chosen should be consistent with the speed a driver is likely to expect. Where a difficult condition is obvious, drivers are more apt to accept lower speed operation than where there is no apparent reason for it."
- "Above minimum design values should be used where feasible, but in view of the numerous constraints often encountered, practical values would be recognized and used."
- "Where it is necessary to reduce design speed, many drivers may not perceive the lower speed condition ahead, and it is important that they be warned well in advance. The changing condition should be indicated by such controls as speed-zone signs and curve-speed signs."

Influences on Operating Speeds

In a 1962 study on operating speeds within the urban environment (*11*), Rowan et al. concluded that substantial speed reductions occurred when sight distance was below 1,000 to 1,200 ft and that the introduction of a curbed urban cross-section and the adjacent land use (residential or commercial development) also had a speed-reduction influence. Lateral restrictions (trees and shrubbery) were found to be a greater speed-reduction influence than development density (*11*). In 1966, Oppenlander reviewed the literature to identify variables influencing spot speed (*12*). The variables were organized into driver, vehicle type, roadway, traffic, and environment categories. The roadway characteristics determined to be most significant included functional classification, curvature, gradient, length of grade, number of lanes, and surface type. Sight distance, lateral clearance, and frequency of intersections were also determined to have an influence (*12*).

Garber and Gadiraju examined speed variances of 36 roadway locations including interstates, arterials, and rural collectors in 1989. Analysis of variance tests were used to determine which traffic characteristics (design speed, highway type, year in which data were obtained, and traffic volume) had a significant effect on average speed and speed variance at the 5 percent significance level (95 percent confidence level). Design speed and highway types were significant while time and traffic volumes were not significant (*13*).

Lefeve (*6*) found that as drivers approach vertical curves with short sight distances, they reduce their speeds to some extent. Where the minimum sight distance was 150 ft, the average reduction in speed was 6 mph. Where the minimum sight distance was 400 ft, the average reduction in speed was only 2 mph. These speed reductions are less than the speed reduction assumed by the AASHTO stopping sight distance model (*2,3*). Lefeve hypothesized that drivers seldom encounter critical situations on vertical curves that they are not aware of the hazard involved and their perception of risk is low and they feel their reduciton in speed is greater than it actually is.

Design Speed and Operating Speed

Recent studies have demonstrated that a noticeable disparity exists between design and operating speeds. A 1992 Federal Highway Administration (FHWA) study measured operating speeds on 50, 60, and 70 mph design speed roadways in three states (*14*). The study results tabulated by site and design speed are shown in Table G-1. Note that the average and 95th percentile speeds for the three design speeds were 55.9, 59.6, and 59.3 mph, and 67.0, 70.5, and 71.0 mph, respectively.

The authors concluded that upper-percentile operating speeds on the 50 and 60 mph design speed roadways have little correlation with the inferred design speed and that the operating speed on these roadways significantly exceeded their design speed. The authors also noted that the largest disparity between design and operating speed (Arkansas 81) occurred on an atypical 50 mph design speed roadway; i.e., full-width paved shoulders and gently rolling terrain. This wider cross section probably contributed to higher operating speeds at this site.

In a 1991 *Public Roads* article on advisory speed setting criteria, Chowdhury et al. reported on speed data for 28 horizontal curves in three eastern states. Figure G-1 illustrates measured 85th percentile speeds and the corresponding inferred design speed for the horizontal curve. The inferred design speed was determined from the standard superelevation equation given the degree of curvature and measured superelevation rate near the midpoint of the curve, and assuming that the AASHTO maximum coefficient of side friction was not exceeded. All curves with inferred design speeds of 50 mph or less had 85th percentile speeds that exceeded the design speed. Only the single 60 mph design speed curve had an 85th percentile speed that was less than the design speed (*15*).

A recent FHWA study on design consistency produced similar results. Speed data were collected at 138 horizontal curves on 29 rural two-lane highways in five states in three geographic regions (*16*). The data, shown in Figure G-2, indicate that the 85th percentile speed exceeded the inferred design speed on all but two curves with design speeds of 55 mph or less. In contrast, the 85th percentile speed was less than the inferred design speed for all curves with design

TABLE G-1. Results of Operating Speed Studies for Three Design Speeds (*14*).

State and Highway	Design Speed (mph)	Sample Size	Average Speed (mph)	95% Speed (mph)
Ark. - 10	50	389	55.8	66.9
Ark. - 81	50	480	59.1	70.4
Ill. - 97	50	887	54.7	66.7
Ill. - 36	50	890	55.6	65.8
Total/Avg	50	2,646	55.9	67.0
Ark. - 70	60	760	58.8	69.3
Ark. - 22	60	464	58.4	68.9
Tex. - 155	60	840	60.7	71.7
Tex. - 21	60	382	60.7	72.4
Total/Avg	60	2,446	59.6	70.5
Ill. - 51	70	950	57.8	68.5
Ill. - 125	70	974	58.5	69.4
Tex. - 79	70	885	59.9	73.7
Tex. - 21	70	754	61.3	73.0
Total/Avg	70	3,563	59.3	71.0

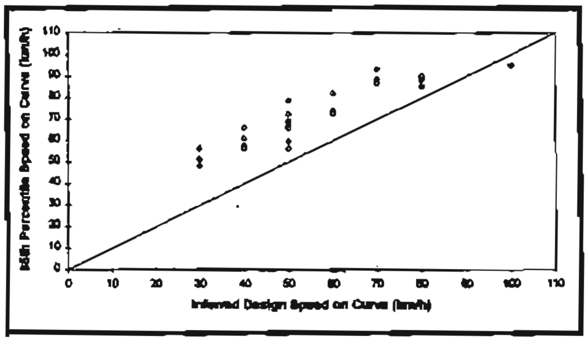

FIGURE G-1. 85th Percentile Speed versus Inferred Design Speed for 28 Horizontal Curves.

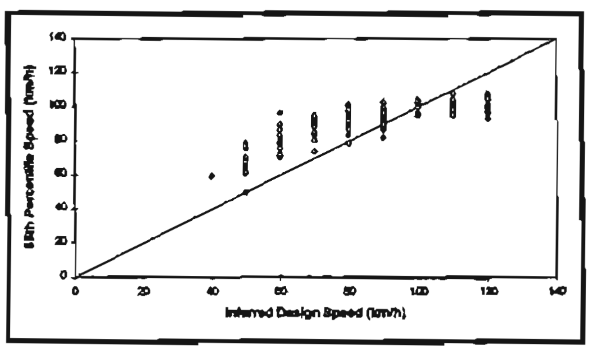

FIGURE G-2. 85th Percentile Speed versus Inferred Design Speed for 138 Horizontal Curves.

speeds of 65 mph or more. For the curves with 60 mph design speeds, an almost equal number had 85th percentile speeds greater than and less than the inferred design speed. The disparity between the 85th percentile speeds and inferred design speeds is greatest for the lowest design speeds. For curves with design speeds between 25 and 40 mph. For example, 85th percentile speeds average 11 to 12 mph faster than the design speed (*16*).

McLean (*9, 17*) found similar design speed/operating speed disparities on rural two-lane highways in Australia. He concluded that horizontal curves with design speeds less than 90 km/h (55 mph) had 85th percentile speeds that were consistently faster than the design speed, whereas curves with design speeds greater than 90 km/h had 85th percentile speeds that were consistently slower than the design speed. McLean's findings prompted a revision of the Australian design procedures for low-design speed roadways.

Design Speed and Speed Limits

Researchers at the Pennsylvania Transportation Institute (*18*) are examining the use of a target operating speed as the preselected design speed for the design of low-speed urban streets. Their goal is to provide street designs that reflect the operating environment so that a complimentary relationship would exist between the preselected design speed, actual operating speed, and posted speed limits. For such a design process to become practical, the ability to predict probable operating speeds along the proposed alinement is required. To do so, the relationships between the probable operating speed and the geometric elements (line, grade, and cross-section), land use, and traffic engineering elements must be determined. One objective of the study is to develop a speed-prediction model for low-speed urban streets.

In the Garber and Gadiraju study of 36 roadway locations where design speed was used as a surrogate for geometric characteristics (*13*), the following was concluded:

- Speed variance tends to be at a minimum when the difference between the design speed and the posted speed limit is between 5 and 10 mph.
- For average speeds between 25 and 70 mph, speed variance decreases with increasing average speed.
- The difference between the design speed and the posted speed limit has a statistically significant effect on the speed variance.
- Drivers tend to drive at increasing speeds as the roadway geometric characteristics improve, regardless of the posted speed limit.
- Accident rates do not necessarily increase with an increase in average speed but do increase with an increase in speed variance.

To reduce speed-related accidents on high speed roadways, the authors recommended speed limits that were 5 to 10 mph lower than the roadway's design speed; however, it should be noted that given a 55 to 65 mph speed limit, their recommendations are nothing more than selecting a design speed that is at or near the roadway's probable operating speed.

Design Speed and Safety Effects

A review of the literature on the safety effects of geometric design yielded some insight regarding the cost-effectiveness of providing longer stopping sight distances. Glennon (*19*) drew the following conclusions from a review of stopping sight distance literature:

- Alignment changes are normally cost-effective only on highways that have very high traffic volumes and major hazards that are hidden by a sight obstruction.
- Treatments such as site-specific warning signs, advisory speed plates, and reduced speed zones should be encouraged at the locations where a crest vertical curve hides a hazard.

Design Speed and Stopping Sight Distance

Sight distance is the length of roadway ahead that is visible to the driver, and as a minimum, the sight distance available on the roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before striking a stationary object in its path. This distance is known as stopping sight distance. The major consideration when designing crest vertical curves is the provision for adequate stopping sight distance.

Although greater length is desirable, sight distance at every point along the roadway should be at least that required for a below average operator or vehicle to stop in this distance. Table G-2 illustrates the changes in stopping sight distance values since the 1940s. Both the minimum and desirable stopping sight distance values are shown for the 1971, 1984, and 1990 publications. A review of Table G-2 shows that stopping sight distance requirements have increased since the 1940s, but that 1984 was the first time that the minimum stopping sight distance requirements were actually increased. Vertical curve length requirements also increased. Table G-3 illustrates the minimum and desirable K-values (length of curve for a 1 percent change in grade) for various design speeds. Again, note that the 1984 and 1990 minimum stopping sight distance requirements and K-values are longer than they were prior to 1984.

The increase in required stopping sight distance resulted from various parameter values within the SSD model changing over time (*20*). Perception-reaction time was changed to a constant 2.5 seconds in 1954. Pavement characteristics originally assumed dry surface conditions; however since 1954, wet surface conditions have been used. Driver eye height has decreased from 4.50 to 3.75 to 3.50 ft because newer passenger cars resulted in lower eye heights. The object height was increased from four to six inches to offset

TABLE G-2. Stopping Sight Distance Since 1940.

Publication	50 mph	60 mph	70 mph
	SSD (ft)	SSD (ft)	SSD (ft)
1940 AASHO	366	480	614
1954 AASHO	350	475	600
1965 AASHO	350	475	600
1971 AASHO*	350-450	475-650	600-850
1984 AASHTO*	400-475	525-650	625-850
1990 AASHTO*	400-475	525-650	625-850

Minimum and desirable values are shown.

TABLE G-3. K-values and Stopping Sight Distances for Various Design Speeds (*2*).

Speed (mph)	Minimum Values		Desirable Values	
	K	SSD (ft)	K	SSD (ft)
25	20	150	20	150
30	30	200	30	200
35	40	225	50	250
40	60	275	80	325
45	80	325	120	400
50	110	400	160	475
55	150	450	220	550
60	190	525	310	650
65	230	550	400	725
70	290	625	540	850

a decrease in driver eye height from 4.5 to 3.75 ft in the 1965 AASHTO Policy; however, the object height was not changed to offset a decrease in eye height from 3.75 to 3.5 ft in the 1984 AASHTO Policy. As a result, slightly longer stopping sight distances were required.

Since the development of the SSD model in the 1940s, the speed used for the model has changed from the design speed of the roadway to a percentage of design speed (as low as 83 percent). The assumed speed for stopping sight distance criteria has historically been less than or equal to the design speed of the roadway because it was thought that motorists drive more slowly on wet pavements than they do on dry pavements and/or that the average running speed is less than the design speed of the roadway. While these speed reductions may have been the case 30 or 40 years ago, they do not appear to be the case today. For example, Lamm, et al. (*21*) found that operating speeds on dry pavements were not significantly different from operating speeds on wet pavements and drivers do not adjust their speeds to accommodate wet pavement on curves. These findings contradict the premise that most motorists drive slower than the design speed of the roadway.

STUDY METHODOLOGY

This section describes the field study design and data analysis procedures used in this research. Roadways with limited stopping sight distance were identified and reviewed to determine their appropriateness for inclusion in the study. Speed data were collected on both tangent and limited sight distance sections, and then analyzed to determine whether motorists drove slower on limited sight distance sections. The following sections describe the site plan information databases, site selection criteria, data collection, and data analysis.

Plan Information Databases

The process of determining potential study sites involved locating rural roadways with limited sight distance vertical curves (curves with K-values less than 150 are those curves that have less than the minimum required stopping sight distance for a design speed of 55 mph). Efforts were focused on older roadways in rolling terrain because there are generally more limited sight distance curves on these types of roadways. To identify potential study sites, information was requested from three state DOTs and FHWA. This information included the following: average daily traffic (ADT), length and radius of horizontal curves, length and grades of vertical curves, number and width of lanes, shoulder width and type, and milepost location of geometric features.

The roadways used in this study were located in three different geographic regions—Washington, Texas, and Illinois. These locations were selected due to the differing geographic conditions they represent, and the quality of the traffic and roadway information databases available from each of the respective agencies. The traffic and roadway geometry information obtained from each of these states are described below.

Washington Sites. Three reports obtained from the Washington Department of Transportation were used in identifying vertical curves with limited sight distance. ADT values were available from the *Annual Traffic Report* while the *Horizontal and Vertical Alignment Report* contained information on: curve location (State Route Milepost); horizontal curves (length, radius, and central angle); and vertical curves (approach grade, departure grade, and curve length). The *State Highway Log Planning Report* contained information concerning number of lanes, shoulder width and type, median type and width, milepost locations of intersections, and speed limit.

Texas Sites. Texas provided a large database of potential sites for this study. Study locations used in previous work done by Fambro et al. (*5*), as well as other locations in the state, were included in the initial evaluation of potential sites. Highway construction plans and the *Texas Road Inventory Report* for each site were obtained from various Texas Department of Transportation districts.

Illinois Sites. A report for Illinois roadways was requested from the Highway Safety and Information Systems (HSIS) computer database maintained by the University of North Carolina. The report contained geometric characteristics (approach and departure grades, length of vertical curve, and milepoint) of individual roadways, as well as ADT information. Information in the form of a database was obtained for all rural limited sight distance vertical curves.

Site Selection Criteria

The plan information was sorted and manipulated so that potential study sites could be identified. Because roadway type and traffic volume may influence operating speeds, the classification scheme shown in Table G-4 was developed to aid in selection of potential study sites that represented three roadway types and three traffic volume levels. Roadway types included multilane roadways (four-lane divided roadways), two-lane with shoulder roadways, and two-lane without shoulder roadways (roadways were categorized as having shoulders if they had paved shoulders six feet or wider) while traffic volume levels included low, medium and high. Traffic volume was included as a study variable to determine whether it had an effect on a driver's desired operating speed. Numeric values for each volume level were chosen after reviewing FHWA's *Highways Statistics* and the Texas *Roadway Inventory Database*. These sources provided information on number of rural miles of roadway by average daily traffic and roadway type. Table G-4 also shows the desired number of study sites in each of the roadway/traffic categories.

A list of potential study sites was identified from the information provided by the states. Criteria for identifying potential study sites were that the roadway had crest vertical curves with less than the AASHTO required minimum stopping sight distance for a design speed of 55 mph (K-value less than 150); that the approach grade was less than 5 percent; that the roadway was in a rural area; that the roadway's cross section and adjacent land use were consistent throughout the site; and that there were no intersections controlled by traffic signals or multi-way stop signs within the site.

Roadways with one or more limited sight distance crest vertical curves were identified by manually inspecting roadway plans in the office. Curves were eliminated as potential study sites if they were on or near a horizontal curve, had a long and/or steep approach grade, or did not have a suitable control section near the curve of interest. The remaining vertical curves and potential control sections were noted along with information on how to locate the individual crest curve locations in the field.

TABLE G-4. Roadway and Traffic Conditions for Daytime Speed Studies.

Type of Roadway	Traffic Volume/Number of Sites			Total
	Low	Medium	High	
Multilane	-	2	2	4
Two-Lane with Shoulders	2	2	-	4
Two-Lane without Shoulders	2	2	-	4
Total	4	6	2	12

Once the list of potential study sites had been developed, a team of two individuals visited each of the sites to determine their appropriateness as actual study sites and to collect additional roadway data (number of driveways, additional cross-sectional information, and roadway characteristics). During the initial visit to the site, each vertical curve within the roadway section was located, and conditions such as neighboring development or other features that could affect operating speeds were identified.

If several potential study sites (crest vertical curves) existed within a section of roadway, the best curve in terms of least number of potential influences on the operating speeds at the control and crest locations was selected as one of the potential study sites. In some cases, all curves along a roadway that were identified in-house were eliminated during the initial site inspection visit, and in other cases, a curve that was not on the plan sheets in our possession was identified as the best crest curve for this study. In this case, the DOT's district office was contacted when the site inspection team returned to the office.

Data Collection

Once the study sites had been identified, speeds were collected for individual vehicles at both a control section and a crest curve section. The speeds on the control section were collected at a location where the vehicles were expected to be operating at their desired speeds, while the speeds on the vertical curve were collected at the point of minimum sight distance; i.e., just before the intersection of the approach and departure grades of the curve. Distances between the control and crest curve section varied from several hundred feet to approximately one mile; however, most of the distances were in the 1,000 to 3,000-ft range. The minimum speed in the limited sight distance section was needed so that the maximum speed differential between the control section and the limited sight distance section could be determined.

A description of each individual vehicle was recorded along with its speed. This description enabled the tracking of the vehicle through both the control and crest curve sections to detect changes in the vehicle's speed. The vehicles also had to fit certain criteria for their inclusion in the study's database. Vehicles that were considered applicable for this study were passenger cars, pickup trucks, or vans. Heavy vehicles were not included because of the possibility of their speeds being affected by length and steepness of grades rather than available sight distance. Vehicles that were included in the study's database had to meet the following criteria:

- Must not have been towing a trailer or other vehicle;
- Must have a headway greater than or equal to five seconds; and
- Must not have entered or exited the roadway within or near the study site.

The rationale for these criteria was that vehicles towing a trailer or other vehicle may have been affected by length or steepness of grade rather than available sight distance. Likewise, vehicles following at a headway less than five seconds may have been affected by the preceding vehicle and a vehicle that has entered the roadway at or near the study site may not have reached its desired operating speed. Furthermore, a vehicle exiting the roadway at or near the study segment may be slowing to turn rather than because of available sight distance. By adhering to these guidelines, it was anticipated that the measured difference in speeds would be a result of limited sight distance rather than traffic conditions along the roadway.

Most speed data were collected with detuned radar guns that were not detectable by commonly used radar detectors. At locations where speeds were collected with normally tuned radar guns, care was used to assure that drivers did not detect the presence of the radar. Data for vehicles that slowed dramatically or drivers that appeared to detect and react to the data collection process were discarded. To avoid distracting drivers, data collectors were positioned in such a way as to make them as inconspicuous as possible. Generally, they were located beyond the right-of-way in lawn chairs behind vegetation or in a parked vehicle.

Data were collected for a minimum of four hours or 100 vehicles. The time limit of four hours eliminated undue delays while collecting data, while the quantity of 100 vehicles provided for a reliable database to perform a statistical analysis. Speed data is generally considered to be normally distributed. Even if it is not, means from large samples have distributions that are normal or nearly normal. With this knowledge, an appropriate statistical analysis of the collected data could be performed.

Data Analysis

After the speed data were collected and reduced, the data were analyzed with a standard statistical software package. The analysis included both analysis of variance tests and regression analysis. The research hypothesis of interest was that there was a reduction or change in speed as a result of the limited sight distance vertical curve and one of the roadway or traffic volume parameters of interest. The following sections present a discussion of both statistical test methodologies that were used.

Analysis of Variance. The data were first analyzed using an analysis of variance test to determine if there was a statistically significant difference in speed reductions as a result of limited sight distance crest vertical curves and one of the roadway type or traffic volume parameters of interest. The analysis of variance tests focused on differences due to type of roadway, traffic volume, design speed and day versus night conditions.

Analysis of variance tests were also used to determine if a statistically significant difference existed between the three states' 85th percentile control speeds. It is important to note that relatively small differences may be statistically significant because of the relatively large sample sizes; however, from a practical standpoint, differences in speed of one or two miles per hour are not meaningful (*22*). Thus, for speed differentials due design speed or available sight distance to be meaningful, they must be statistically significant as well as large enough to be practically significant.

Regression Analysis. The data were then analyzed using regression analysis to determine if there was a statistically significant relationship between design and 85th percentile speeds for the crest vertical curves in the database (i.e., is crest vertical curve geometry a good predictor of 85th percentile operating speeds). The method of least squares regression analysis determines an equation for predicting 85th percentile speeds given the design speed of the curve. In this case, the design speed is the independent variable that will be used to predict values of the crest curve's 85th percentile operating speed. The parameter estimates and the standard error of the estimates can then be used to determine the statistical significance of the relationship or regression equation.

RESULTS

A statistical comparison was performed between speeds at the control and crest sections for various roadway types, traffic volumes, and design speed. A comparison was also performed between states to determine any differences in 85th percentile speeds for different roadway types. Finally, a regression analysis was performed to determine if design speed is a good predictor of 85th percentile speed.

Parameters Studied

The objective of this study was to determine if there is a difference in the operating and design speeds of limited sight distance crest vertical curves and if so, are their differences in speed reductions due to changing traffic and roadway parameters? Thus, the basic research hypothesis of interest was whether drivers reduced their speed on limited sight distance vertical curves. Three roadway types (multilane, two-lane with shoulders and two-lane without shoulders) and three traffic volume levels (low, medium, and high) were defined for this study.

Within these nine roadway-type, volume-level categories, 42 operating speed studies were conducted at 39 sites in three states (data were collected twice at three sites—once during the day and once during the night). More studies were conducted than planned because the closeness of the study sites facilitated additional data collection with minimal additional costs. Table G-5 provides a breakdown of the 42 studies in each of the nine study categories. Note that with the exception of one of the high volume categories there were at least two sites in each of the categories studied.

Besides the roadway type and traffic volume level, each site was further subdivided by the inferred design speed of the vertical curve where data were collected. Design speed categories are analogous to available sight distance at the crest vertical curve and were set at Level 1—50 to 55 mph (400 to 450 ft of available sight distance), Level 2—40 to 49 mph (275 to 400 ft of available sight distance), and Level 3—40 mph and below (275 ft or less of available sight distance). Table G-6 illustrates the 27 categories (3 roadway types x 3 traffic volume levels x 3 design speeds) in the final study design.

Preliminary Analysis

As a first step in the analysis process, the mean reduction in speeds between the control and crest curve sections were calculated for each of the 42 studies where data were collected. These data are summarized by site in Table G-7. When examining a speed study site matrix, it shows that several cells contain 0 or 1 site. These cells do not contain more sites because in the field, certain combinations of conditions, such as multilane and two-lane with shoulder roadways with low design speeds and very short sight distances, do not exist or are very few in number.

The mean reductions in speeds for the different design speed categories were then plotted (Figures G-3, G-4, and G-5) for each of the three roadway categories to gain initial insight in the relationship between design and operating speed. Knowing that lower design speed crest curves provide less sight distance than higher design speed curves, roads with lower design speed curves were expected to have a larger reduction in speeds between the control and crest sections. In other words, drivers were expected to decrease their speed in response to decreasing amounts of sight distance.

Examination of Figures G-3 and G-4 (multilane roadways and two-lane roadways with shoulders) supports this premise by showing larger reductions in speed with decreasing amounts of sight distance; i.e., lower design speeds. For some conditions, there was also a slight tendency for smaller reductions in speed with increasing traffic volumes and for other conditions, there was a slight tendency for larger reductions in speed with increasing traffic volumes. Figure G-5 (two-lane roadways without shoulders) shows larger reductions in speed with increasing traffic volumes for the two higher design speeds, but the opposite trend for the lower design speed. The two-lane without shoulder roadway that appeared inconsistent with the other observations was from the low volume, less than 40 mph design speed study site.

TABLE G-5. Study Sites by Roadway Type and Traffic Volume.

Roadway Type	Traffic Volume Levels/ Number of Sites			Total
	Low	Medium	High	
Multilane	-	5	3	8
Two-Lane With Shoulders	7	6	1	14
Two-Lane without Shoulders	6	10	4	20
Total	13	21	8	42

TABLE G-6. Study Sites by Roadway Type, Design Speed and Traffic Volume.

Roadway Type	Design Speed (mph)	Traffic Volume		
		Low	Medium	High
Multilane	50-55 40-49 ≤ 40	0-5,000	5,000-10,000	Over 1-0,000
Two-Lane With Shoulders	50-55 40-49 ≤ 40	0-3,000	3,000-5,000	Over 5,000
Two-Lane Without Shoulders	50-55 40-49 ≤ 40	0-3,000	3,000-5,000	Over 5,000

TABLE G-7. Mean Reduction in Speeds by Design Speed and Traffic Volume.

Roadway Type	Design Speed (mph)	Traffic Volume		
		Low	Medium	High
Multilane	50-55	-	TX 6 - 2.3 (day) TX 6- 2.2 (night)	IL 29 - 1.3
	40-49	-	TX 69 - 4.9 IL 51 - 1.3 IL 28 - 2.3	TX 31 - 3.6 IL 12 - 4.5
	< 40	-	-	-
Two-Lane With Shoulders	50-55	IL 23 - 0.4 WA 97 - 2.1 WA 97 - 0.9 WA 101 - 1.2	-	WA 101 - 1.4
	40-49	WA 14 - 3.1 WA 410 - 2.8	IL 127 - 1.2 TX 80 - 3.3 TX 80 - 0.8 TX 64 - 3.4 TX 64 - 3.7 WA 101 - 3.9	-
	< 40	IL 116 - 3.1	-	-
Two-Lane Without Shoulders	50-55	IL 72 - 3.7 TX 19 - 0.5 WA 97 - 2.3	TX 14 - 2.3	WA 101 - 1.2 (day) WA 101 - 1.5 (night)
	40-49	TX 3058 - 3.8 IL 29 - 1.3	TX 14 - 6.6 TX 1179 - 1.7 TX 1179 - 3.2 WA - 3.3 WA 203 - 0.4	IL 173 - 3.4 WA 507 - 0.1
	< 40	*WA 7 - 1.8	IL 20 - 5.3 (day) IL 20 - 4.4 (night) TX 315 - 2.1 WA 14 - 3.5	-

* *Not used in analysis due to somewhat unusual geometry at the study site.*

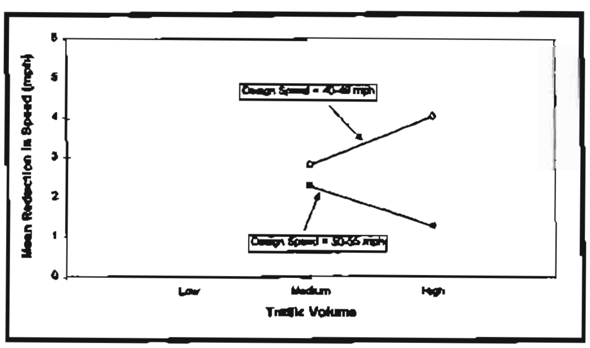

FIGURE G-3. Mean Reduction in Speeds for Multilane Roadways.

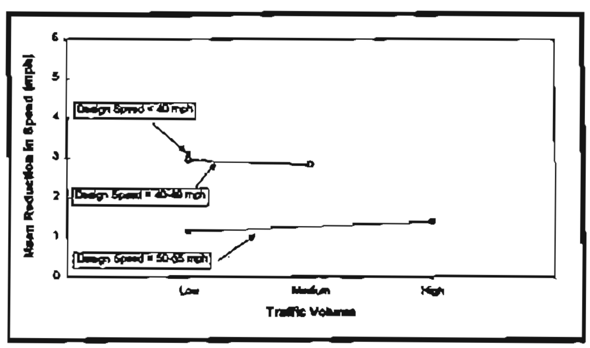

FIGURE G-4. Mean Reduction in Speeds for Two-Lane Roadways with Shoulders.

Further investigation of this data point indicated that the the inconsistencies were caused by data from a single study site (WA 7). The mean reduction in speeds between the crest and control sections on WA 7 was 1.8 mph; however, based on the shortness of available sight distance at this sight, a larger reduction in speeds was anticipated. This discrepancy was probably the result of the control being on a slight downgrade, the approach to the crest being a relatively flat 0.5 percent upgrade and the distance between the control and crest being 650 ft; i.e., the short distance between the control and crest sections made it difficult, if not impossible, for larger reductions in speed to occur.

To avoid the possibility of reaching erroneous or at best confusing conclusions, data from this site were removed from the analysis and the mean reductions in speeds for the remaining sites were plotted as shown in Figure G-6. A review of the mean reduction in speeds in the revised plot shows that as the available sight distance decreases (lower design speeds), the mean reduction in speeds increases. This observation is consistent with the other roadway type categories; i.e., the more limited the sight distance, the more drivers reduced their speed. If drivers associate risk with available sight distance, they may be decreasing their speed in response to an increase in percent risk.

Analysis of Variance

Confidence levels indicate how confident one is that observed differences are the result of the factors of interest and not the result of chance; i.e., reductions in speed are a result of study treatments and not chance. For this study, the level of significance chosen was 0.05, which translates to a 95 percent confidence level. Stated another way, statistical significance would indicate that one is at least 95 percent confident that drivers are slowing because of roadway, traffic, and/or geometry conditions.

Multilane Roadways. The mean reduction in speeds between the control and crest section for each design speed and volume level were plotted for the multilane roadway category in Figure G-3. This figure showed that as the design speed (available sight distance) decreased, the mean reduction in speeds between the control and crest sections increased. It also showed that as the traffic volume increased, the mean reduction in speeds increased for one design speed category and decreased for the other design speed category; however, it should be noted that there were data for only four of the nine cells in this roadway category.

Because of the missing cells in this roadway category, only two hypotheses could be tested: differences due to traffic volume and differences due to design speed. In the first set of tests, it was concluded that the differences in mean reduction in speeds between medium and high volumes in the 50 to 55 mph design speed category were not large enough to be statistically significant, and that the differences in mean reduction in speeds between medium and high volumes in the 40 to 49 mph design speed category were large enough to be statistically significant. Thus, in one design speed category, increases in traffic volumes resulted in smaller reduction in speeds between control and crest locations, and for the other design speed category, increases in traffic volumes resulted in a significantly larger reduction in speeds; however, in both cases, the magnitude of these differences was not large enough to be meaningful.

In the second set of tests, it was concluded that the differences in mean reduction in speeds between the 50 to 55 and 40 to 49 mph design speeds in the medium volume category were not large enough to be statistically significant, and that the differences in mean reduction in speeds between the 50 to 55 and 40 to 49 mph design speeds in the high volume category were large enough to be statistically significant. In other words, decreases in design speed and available sight distance, caused a significantly larger reduction in speeds between control and crest sections for high volume sites, but not for medium volume sites. In this study, the magnitude of the difference in reduction in mean speeds for the high volume study sites was approximately 2.7 mph.

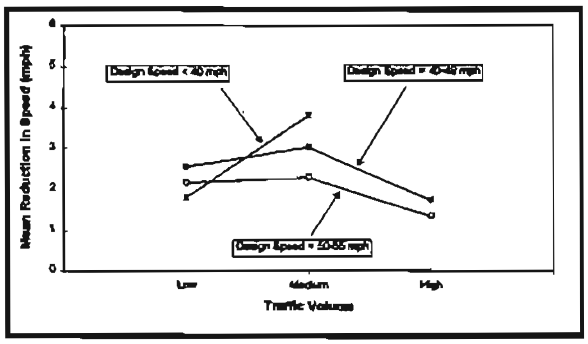

FIGURE G-5. Mean Reduction in Speeds for All Two-Lane Roadways without Shoulders.

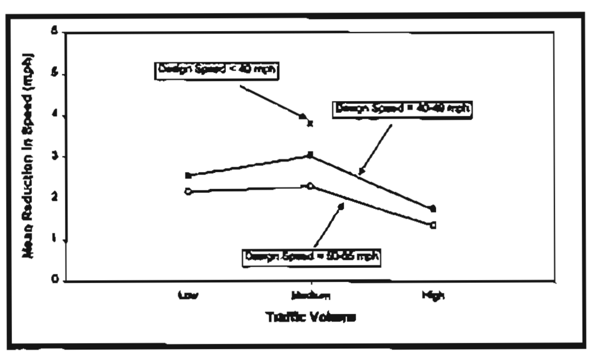

FIGURE G-6. Mean Reduction in Speeds for Remaining Two-Lane Roadways without Shoulders.

Two-Lane Roadways with Shoulders. The mean reduction in speeds between the control and crest section for each design speed and volume level were plotted for the two-lane roadways with shoulder category as shown in Figure G-4. As with the multilane roadways, the mean reduction in speeds between the control and crest sections increased as design speed and available sight distance decreased. There did not appear to be any effect on mean reduction in speeds due to changes in traffic volumes. Several hypotheses were tested.

The first two comparisons performed involved single design speed categories and multiple volume levels. In the first test, results indicated that the differences in mean reduction in speeds between low and high volumes in the 50 to 55 mph design speed category were not large enough to be statistically significant. In the second test, it was concluded that the differences in mean reduction in speeds between low and medium volumes in the 40 to 49 mph design speed category also were not large enough to be statistically significant. Thus, increases in traffic volumes on two-lane roadways with shoulders do not appear to have an effect on mean reductions in speeds between control and crest sections.

The last comparison performed involved a single traffic volume category and multiple design speed categories. It was concluded that the differences in mean reduction in speeds between the 50 to 55 mph design speed, 40 to 49 mph design speed, and less than 40 mph design speed categories were large enough to be statistically significant. It was also concluded that the mean reduction in speeds for the 50 to 55 mph design speed category was significantly different from the two lower design speed categories. Thus, decreases in design speed and available sight distance on low volume two-lane roadways with shoulders caused larger reductions in speed between the control and crest sections.

Two-Lane Roadways without Shoulders. The mean reduction in speeds between the control and crest section for each design speed/volume level for the two-lane roadways without shoulders were plotted Figure G-5. As noted previously, one of the sites in this roadway category had some unusual characteristics which could have resulted in confusing or misleading results. To minimize the possibility of erroneous conclusions, this data point was deleted from the analysis data set and the mean reduction in speeds plotted as shown in Figure G-6. As with the other two roadway categories, the mean reduction in speeds between the control and crest sections increased as design speed and available sight distance decreased. It also appears that mean reduction in speeds decrease with increasing volumes. Several hypotheses were tested.

The first two comparisons performed involved single design speed categories and multiple volume levels. In the first test, it was concluded that the differences in mean reduction in speeds between low, medium, and high volumes in the 50 to 55 mph design speed category were large enough to be statistically significant. In the second test, it was concluded that the differences in mean reductions in speeds for low, medium, and high volumes in the 40 to 49 mph design speed category also were large enough to be statistically significant. Thus, increases in traffic volume appear to have a statistically significant effect on mean reductions in speed for two-lane roadways without shoulders; however, the magnitude of this effect is not large enough to be meaningful.

The next three comparisons performed involved single volume levels and multiple design speed categories. In the first test, it was concluded that the differences in mean reductions in speeds between the 50 to 55 mph and the 40 to 49 mph design speed categories for the low volume category were not large enough to be statistically significant. In the second test, it was concluded that the differences in mean reduction in speeds between the 50 to 55 mph, 40 to 49 mph, and less than 40 mph design speed categories for the medium volume category were large enough to be statistically significant. In the third test, it was concluded that the differences in mean reductions in speeds between the 50 to 55 mph and 40 and 49 mph design speed categories for the high volume category were not large enough to be statistically significant. Thus, decreases in design speed and available sight distance resulted in larger speed reductions between control and crest sections; however, the magnitude of this increase tends to diminish with increases in traffic volumes.

Day/Night Comparisons. Analysis of variance tests were conducted to determine whether there were statistically significant differences between mean reduction in speeds for day and night driving conditions. As shown in Table G-7, day and night data were collected for one multilane roadway site and two two-lane without shoulder sites. Mean reduction in speeds were then found for each site and time period, and plotted as shown in Figure G-7.

Note that the differences in mean reduction in speeds between day and night conditions at two of the sites were similar, but that the third site had a noticeably larger reduction in speeds between day and night conditions. Interestingly, the two similar sites were the multilane site and one of the two-lane sites, rather than the two two-lane sites. The probable explanation for this result is that the two similar sites both were in the 50 to 55 mph design speed category whereas the third site, a two-lane roadway without shoulders, was in the less than 40 mph design speed category. The larger speed reduction at the latter site is consistent with other comparisons which show that the largest reductions in speeds between control and crest sections are associated with those crest curves that have the lowest design speeds and shortest sight distances.

The combined results of the analysis of variance tests support the work by Lefeve (*6*). Lefeve determined that as sight distance was decreased, differences in vehicles' speeds between the control and limited sight distance sections increased (2 mph for 400 ft of sight distance and 6 mph for 150 ft of sight distance). This reduction in speed, however, is much less than AASHTO's current assumptions for determining minimum stopping sight distances (6 mph for 12 400 ft of sight distance, i.e., a 45 to 50 mph design speed).

Between State Comparisons. Analysis of variance tests were also performed to determine if there were statistically significant differences between the mean 85th percentile speeds from the three states. In other words, are there differences in the speed that motorists drive on multilane, two-lane with shoulder, and two-lane without shoulder roadways due to geographic location. Because speeds for multilane roadways were collected in only two of the states (Illinois and Texas), they were not included in the between state comparisons.

The comparisons performed involved the 85th percentile speeds at the control sections of two-lane roadways in each of the three states. The control sections were generally level segments located within a mile of the crest vertical curves. In the first test, it was concluded that the differences between 85th percentile speeds for the two-lane with shoulder roadways in the three states (62.3, 61.3, and 61.1 mph) were not large enough to be statistically significant. In the second test, it was concluded that the differences between 85th percentile speeds for the two-lane without shoulder roadways in the three states (62.0, 60.6, and 59.8 mph) also were not large enough to be statistically significant. Thus, there did not appear to be a difference in 85th percentile speeds on the control sections for either type of two-lane roadway in the study's database.

Operating and Design Speed Comparison. To compare operating speeds with inferred design speeds, 85th percentile speeds for each of the 39 crest curve sections were computed and plotted versus their corresponding design speed as shown in Figures G-8 and G-9. Data points are located at the intersection of the 85th percentile speed and the inferred design speed of the crest vertical curve. The diagonal line represents those points where the 85th percentile speed at the crest equals the inferred design speed of the vertical curve. The difference between the 85th percentile speed and inferred design speed of the crest curve was greater for the lower design speeds and smaller for the higher design speeds; however, the difference at the higher design speeds is still rather large. Note that it appears that 85th percentile speeds would be less than the design speed at design speeds greater than about 65 mph.

These results are consistent with the literature. Messer (*14*) found that 85th percentile operating speeds were higher than the design speed of 50 and 60 mph design speed roadways and less without shoulders) to determine if design speed is a good predictor of 85th percentile crest speeds. The results of these tests are summarized in Table G-8. Note that in both the multilane roadway and two-lane with shoulder roadway categories, the level of significance of the regression coefficient is less than 95 percent. This result indicates that in the range of conditions studied, the inferred design speed (available sight distance) of the crest curve is not a good predictor of 85th percentile speeds for these types of road ways. In other words, the 85th percentile speeds are not changing as the available sight distance of the crest curve changes on multilane and two-lane with shoulder roadways.

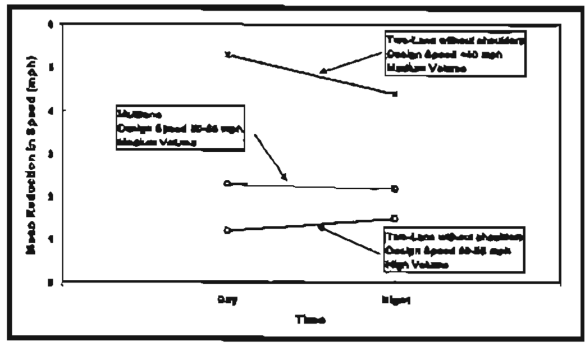

FIGURE G-7. Mean Reduction in Speeds Between Day and Night Operations.

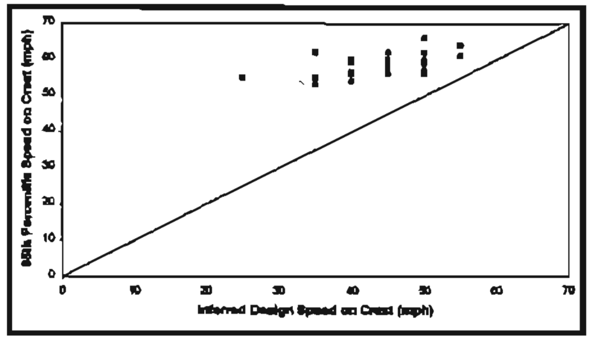

FIGURE G-8. 85th Percentile Crest Speeds for Various Design Speeds (Day Operations).

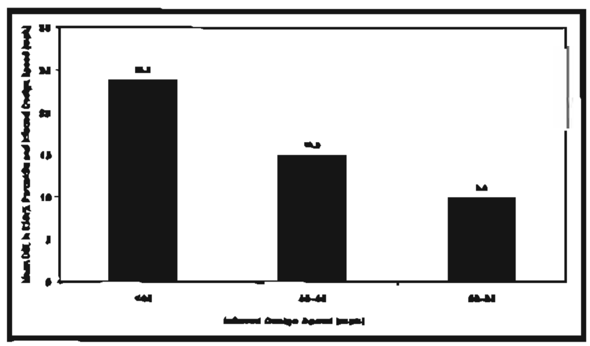

Figure G-9. Mean Difference in 85th Percentile Inferred Design Speed.

Table G-8. Summary of Regression Analysis for 85th Percentile Crest Speeds.

	Multilane	Two-Lane with Shoulders	Two-Lane without Shoulders
Number of Observations	3	14	17
y-intercept (β_0)	-9.00	57.05	45.21
Slope of Prediction Line (β_1)	1.50	0.05	0.29
Coefficient of Determination (R^2)	0.99	0.02	0.48
Level of Significance	0.93	0.34	0.99

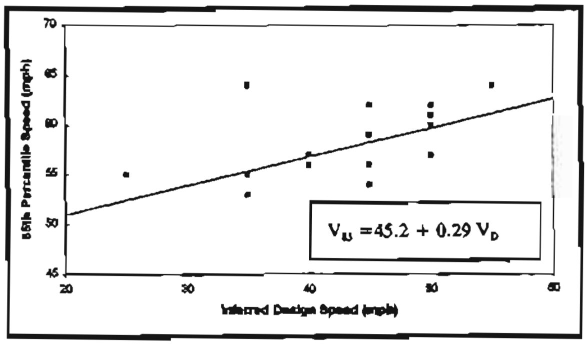

FIGURE G-10. Regression Analysis - 85th Percentile Crest Speeds for Various Design Speeds (Two-Lane Roadways without Shoulders).

In the two-lane without shoulder roadway category, the regression coefficient was significant at the 99 percent confidence level with a coefficient of determination of 0.48. This result indicates that the inferred design speed (available sight distance) of a crest curve is a moderately good predictor of 85th percentile crest speeds for these types of roadways. In other words, the operating speed selected by drivers varies with the available sight distance at crest curves on two-lane without shoulder roadways; however, it should be noted that 85th percentile speeds are greater than crest curve design speeds for all design speeds less than about 65 mph. Figure G-10 shows the plot for 85th percentile and inferred design speeds, and the regression equation for the two-lane without shoulder roadway category.

A somewhat perplexing question is why drivers appear to adjust their operating speed in response to changes in design speed and available sight distance for two-lane without shoulder roadways, but do not appear to adjust their operating speed in response to changes in design speed for two-lane with shoulder roadways? The primary differences in these two types of roadways are the generally wider clear zones and rights-of-way associated with the two-lane with shoulder roadways. It is thought that the paved shoulder and the generally less restricted roadside lessen the perceived risk to the driver and there is not an operating speed adjustment due to changes in available sight distance. On the other hand, the generally more restricted roadside associated with two-lane without shoulder roadways increases the perceived risk to the driver and causes an operating speed adjustment due to changes in design speed.

This premise that perception of hazard or risk influences a driver's operating speed agrees with McLean's findings (*4*). From a practical standpoint, this finding indicates that substantial improvements and/or widening of a roadway's cross section may alter a driver's perceived risk and inadvertently increase the roadway's 85th percentile speed.

CONCLUSIONS AND RECOMMENDATIONS

The operating speed studies documented in this appendix examined the relationship between design and operating speeds for crest vertical curves with limited sight distance. The questions studied included the reduction in operating speeds between control and crest sections, and the relationship between operating and design speeds on crest vertical curves. The reductions in speeds were compared to identify significant differences in operating speeds for various roadway types, traffic volumes, and design speed levels. Conclusions and recommendations are discussed in the following sections.

Conclusions

- For the range of crest vertical curve designs studied (30 to 55 mph design speeds), both the 85th percentile and the average (50th percentile) operating speeds were well above the design speeds of the crest vertical curves.

- For the range of crest vertical curve designs studied, the data suggest that the lower the design speed the larger the difference between the 85th percentile speed and the design speed.

- Available sight distance appears to influence the reduction in speeds between the control and crest sections. As available sight distance is decreased, the mean reduction in speeds between the control and crest sections tend to increase; however, the reduction in speed is less than that suggested by the current AASHTO criteria.

- For the range of conditions studied, traffic volume appeared to have little influence on the mean reduction in speeds between the control and crest sections.

- For the range of conditions studied, roadway type appears to have little influence on the mean reduction in speeds between the control and crest sections.

- For the range of conditions studied, there was not a significant difference between 85th percentile control speeds for the three states in this study.

Recommendations

- Based on the results of this study, it is recommended that the AASHTO stopping sight distance values be based on a speed that is equal to the design speed of the roadway. Minimum values for design should **not** be based on speeds that are less than the design speed of the roadway.

- Because the 85th and 50th percentile crest speeds were above the corresponding design speeds for all of the crest vertical curves in this study, a method to check consistency between operating and design speeds should be developed. AASHTO should also consider revising its guidelines for selecting design speeds so that they encompass a large majority of the expected driving population.

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APPENDIX H -

TORT LIABILITY ISSUES

Liability lawsuits against governmental entities for inadequate or insufficient stopping sight distances are typically based on alleged negligence, or a tort. A tort is a "private or civil wrong that results in injury or loss" (*1*). Typical claims against a governmental entity (frequently a state Department of Transportation) are that they designed the roadway improperly, they set the speed limit too high, or vegetation or construction activity restricted sight distance. To be successful, the plaintiff must prove that the governmental agency failed to provide the necessary sight distance or appropriate warning, that the failure to provide the necessary sight distance or warning caused injury, and that negligent behavior was the reason for the failure.

Tort liability lawsuits against states and other governmental entities have been rising at an alarming rate during the last 20 years. This appendix discusses tort liability as it relates to stopping sight distance. General tort liability issues will be identified and discussed in detail. Specific stopping sight distance issues also will be addressed, including "liability" concerns and potential problems for the future. Finally, the results of a telephone survey of the State Departments of Transportation (DOTs) will be presented.

NEGLIGENCE AND LIABILITY

Liability and responsibility are essentially synonymous. When an individual or governmental entity is liable for an activity, that individual (or governmental entity) is responsible for that activity. If the responsible party fails to perform or improperly performs that activity, and the failure of that activity results in the injury or death of another party, the "injured" party may face legal action initiated by the "wronged" party (*2*).

Public entities and their employees have been given the authority to construct and maintain streets and highways and regulate motor vehicle movements on those roadways by motor vehicle codes and ordinances. Consequently, public entities and their employees are given the responsibility (and inherit the liability) for those specific roadways within each entity's jurisdiction. They are also liable for any improper action taken while designing, operating, and maintaining their streets and highways.

Governmental entities have a duty to operate and maintain those roadways in a "reasonably safe" condition. This duty includes conforming to agency design standards and guidelines, agency policies and procedures, and recognized engineering principles and practices. If the governmental entity fails to provide a "reasonably safe" condition and injury or loss results from this failure, the entity may be liable for that injury or loss.

A liability lawsuit resulting from a traffic accident is founded in the area of law called tort law; therefore, a lawsuit filed because of a traffic accident is usually referred to as a tort case or a tort liability case. The most common tort case involves alleged negligence. A major problem associated with addressing tort cases is determining what is negligence and what is "reasonably safe."

Negligence is defined as "failure to exercise the care that a prudent person usually exercises, or carelessness shown in neglect." **Prudent** is defined as "able to govern and discipline oneself by the use of reason, or exercising caution as to danger or risk." **Reasonable** is defined as "not extreme or excessive; rational; possessing sound judgment" (*3*).

The above definitions should not be considered precise or exact but are certainly realistic. The difficulty presented to judges and juries in tort cases is not to define those terms precisely, but to apply their definitions to the actions of the parties involved in a lawsuit; i.e., the definitions are very subjective. What may be reasonable and prudent to one individual may not be reasonable and prudent to someone else.

In a typical lawsuit, the individual making the complaint (the plaintiff) accuses the individual or organization (the defendant) of negligence. In criminal law, the accuser (the plaintiff) is typically the government and the wrongdoer (the defendant) is the alleged criminal. The roles are typically reversed in tort law because the government becomes the alleged wrongdoer or the defendant.

ELEMENTS OF A TORT LAWSUIT

If a traffic accident occurs and someone is injured, the injured party (or parties) may sue the public entity responsible for the roadway. The plaintiff must prove five basic elements to "win" the case. Those elements are the following:

1. **The defendant has a duty to the injured.** The governmental entity is responsible for the design, construction, and maintenance of streets and highways. If an alleged roadway deficiency caused the accident to occur, showing that the defendant had a duty to the injured party is not difficult.

2. **There was a breach of that duty.** If the roadway "defect" can be shown to be caused by the negligent actions of the governmental entity or its employees, then a breach of the entity's duty could be shown.

3. **The breach of duty was the proximate cause of the injury.** Though a roadway deficiency may exist at the time of the accident, the plaintiff must prove that the defect was the primary cause of this accident. The proximate cause is the legally identified cause of the accident.

4. **There was minimal or no contributory negligence on the plaintiff's part.** In some states, if the plaintiff contributed half or more to the cause of the accident, the plaintiff cannot receive any compensation from the defendant. In other states, the plaintiff can receive the portion (percentage) of the award assigned to the defendant, even if the defendant was found less than 50 percent responsible for the accident.

5. **There must be injury or loss.** If the case is heard in court, the judge and/or jury must find that the plaintiff suffered injury or loss before compensation can be given to the plaintiff.

In addition to these five elements, the plaintiff must prove that a roadway defect or hazardous condition actually existed, that the defect was the proximate cause of the accident, and that the defendant had actual or constructive knowledge of the defect or hazardous condition. **Actual** knowledge means that the defendant was officially informed of the defect in some manner. **Constructive** knowledge means that the defendant should have known of the defect because of previous experiences, existing records in possession of the defendant, or necessary inspections of its roadway system. The defendant's objective is to convince the court (judge and jury) that there was no defect, and if there were a defect, it did not cause the accident to occur, and/or that it was not aware of the defect or hazardous condition.

Tort law is not simply a case of black and white issues. Much "gray" area remains that requires opinions, arguments, evaluations, and judgments. Courts have attempted to be reasonable and fair to both plaintiffs and defendants although individual case decisions may not be considered fair by the losing party. The following guidelines have been developed by courts to help in making decisions concerning responsibility and negligence (*2*)

- The governmental entity is not an insurer of its roads or a guarantor of absolute safety of the motoring public.
- Motorists have a right to expect or presume that the roadway on which the driver is traveling is safe for ordinary use during daytime and nighttime conditions. If unique or extraordinary obstructions, defects, or impediments are present, then appropriate warnings should be provided.
- The motoring public expects the roadway to be safe for travel under "normal" conditions, according to typical and acceptable engineering standards and practices.
- In maintaining a roadway for reasonably safe operating conditions, wide latitude exists for exercising discretionary decisions. Discretionary decisions are decisions made at the discretion of an engineer or technician choosing between logical and valid alternatives. More simply, it is the selection of one alternative from several valid alternatives.
- Certain factors affect the ability to maintain roadways in reasonably safe conditions, including the weather, terrain, and available construction materials.
- The mere presence of a roadway defect does not prove negligence and provide a basis for recovery of damages. Negligence by the defendant must be proven.
- Negligent behavior by the defendant can be described as "having knowledge of a dangerous or defective condition on a roadway and failing to safeguard the public from that condition."

RESPONSIBILITIES OF PUBLIC ENTITIES

If a defective condition exists on a roadway, the public entity responsible for that roadway has certain responsibilities. After noticing the defect, whether it is actual or constructive observation, the public entity has a duty to either correct the defect, protect the public from the defect, or warn the public of the defect. The appropriate action depends on the type of defect, the ability (because of time, material, labor, and financial resources) to correct the problem, and to some extent, the severity of the defect. No defect should be ignored but in certain circumstances (like natural disaster), priorities may be established to decide which defects receive immediate attention.

At one time governmental entities enjoyed the protection of sovereign immunity and could not be sued. Some states continue to enjoy this protection; however, most states have lost a portion or all of their sovereign immunity status. In most states, some protection of sovereign immunity for roadway design or for discretionary decision-making still exists. Though this immunity may be established by law, the public entity may still be challenged with a claim of design error or abuse of discretionary freedom. For instance, some courts have found governmental entities negligent for choosing designs that did not satisfy design standards or guidelines that were in effect or known at the time the roadway was designed. Though rare, blatant disregard of design standards or acceptable design features are obvious examples of the abuse of design discretion.

GOVERNMENTAL DUTIES RELATED TO STOPPING SIGHT DISTANCE

One of the most basic design elements is sight distance. As a minimum, motorists must be provided appropriate stopping sight distances as they travel along a roadway. Occasional provision of passing sight distance is a necessity on most two-lane, two-way roadways. Appropriate sight distance at intersections is important for safe operation of vehicles entering intersectional areas, especially on high-speed roadways.

Regarding sight distance and tort liability, most claims of insufficient sight distance relate to intersection sight distance. Few cases pertain to passing sight distance unless improper markings or signing was present. Stopping sight distance was considered a relatively infrequent tort issue because of several issues.

- Stopping sight distance has been a significant design feature since the 1930s; therefore, most of the roadways in this country were designed with sufficient stopping sight distance.
- Design stopping sight distance is based on the ability of a driver to see a small object on the roadway. Typically, a traffic accident involves a vehicle striking another vehicle or traveling off the roadway. Stopping sight distances that allow drivers a line of sight to small objects result in longer lines of sight and stopping sight distances (at the same operating speed) for large objects.
- Since the development of design guidelines in the 1930s, design criteria changed more significantly in the areas of roadway width, provision and width of shoulder, and roadside safety features. Only minimal changes were made to stopping sight distances.

TELEPHONE SURVEY

An effort was made to contact the Office of the Attorney General or legal counsel for the DOT each in each state to identify specific stopping sight distance issues in the defense of tort lawsuits against the states and to determine the states' exposure to limited stopping sight distances. The intent of the survey was to identify specific issues that should be addressed in more detail in the analysis of current stopping sight distance values and in the assumptions made in determining those values. The secondary purpose of the survey was to identify the potential effect on tort liability if longer stopping sight distances were recommended for new roadway design and/or retrofitting existing roadways. Finally, the survey should raise the awareness of potential exposure to liability that the states might face if longer stopping sight distances were recommended. The results of the survey provided remarkably consistent findings.

As mentioned, an effort was made to contact each state by telephone. While unsuccessful in obtaining information from every state, responses were obtained from 34 states. Responses were received from attorneys, engineers, and risk managers from the DOTs or Offices of Attorney General. The findings from the telephone survey can be summarized by the following points:

- The majority of respondents could not recall any cases dealing with the issue of stopping sight distance. Frequently, plaintiff accusations included an allegation of inappropriate sight distance but such claims were quickly dismissed for lack of evidence and became a non-issue. Some state respondents recalled maybe one or two cases within the past five years that dealt with stopping sight distance.
- Those respondents that indicated some experience with stopping sight distance in tort lawsuits stated that such cases represented less than 1 percent of all cases; hence, the issue was of little concern.
- None of the respondents could provide an example of case law addressing any issue involving stopping sight distance.
- When asked to identify specific details of alleged stopping sight distance deficiencies, it became obvious that most of those cases involved driveways (intersection sight distance), inclement weather (especially fog), or gross inattentiveness (stopping sight distance was actually not an issue). One example that stands out was the complaint of insufficient stopping sight distance on a roadway even though the driver of the rear-ending vehicle admitted being distracted by a coyote immediately prior to impact.
- Legitimate cases of stopping sight distance lawsuits involved either construction zone blockages, improperly posted speed limits, or design error.
- Not surprisingly, none of the respondents believed that a reduction of design stopping sight distances would cause problems in defending tort lawsuits.
- The most revealing and helpful responses came from the question inquiring of the expected problems or liability concerns that the states would experience if design stopping sight distances were increased. These answers deserve more extensive explanations.

Several respondents indicated that an increase in design stopping sight distance should be recommended if the results of the study indicated that additional stopping sight distances were needed to provide safe conditions for motorists. If technological advances or experience indicates that changes are required, the changes should be made for the benefit of the motoring public; however, change for the sake of change would not be desirable. Anytime that design criteria change such that additional requirements are placed on states, then accusations of "inferior" conditions will certainly follow in tort lawsuits.

Most respondents believe that design immunity would provide protection from future roadway design criteria changes. In other words, as long as the state designed a roadway in accordance with the design standards or appropriate design criteria in effect at the time of design, then design alterations or revised design standards would not place the state at risk for negligent design. The few respondents currently struggling with design immunity issues were much more concerned with any changes to current design criteria, believing that liability exposure in the individual's state would increase.

The one concern identified as a potential problem if design stopping sight distances were increased dealt with existing (and typically very old) roadways. In the past, design stopping sight distances were shorter than those recognized and accepted today. These values have lengthened due to the use of shorter driver eye heights and arbitrarily selecting lower design coefficients of friction.

As a result, older roadways designed for minimum stopping sight distances at 55 mph currently have design stopping sight distances that would satisfy design speed. Thus, many people mistakenly believe that the older roadway "has a lower design speed" than when it was built. A more correct statement would be that the older roadway "has a lower inferred design speed." In reality, the recommended design criteria changed rather than the older roadway's design speed.

SUMMARY

Stopping sight distance is not an existing tort liability problem in any state. Nonetheless, increasing required stopping sight distances will increase the exposure of states to future tort liability issues, especially on older roadways that were constructed to minimum design requirements. Any increase in required stopping sight distances should be justified based on sound technological and safety reasons.

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3. Webster's Seventh New Collegiate Dictionary, G. & C. Merriam Company, Springfield, Massachusetts, 1970.

APPENDIX I

RECOMMENDED REVISIONS TO THE AASHTO GREEN BOOK

This appendix presents recommended revisions to the design policies in the AASHTO publication, A Policy on Geometric Design of Highways and Streets, known as the AASHTO Green Book (*1*). The Green Book is the primary geometric design guide used by many transportation departments and other geometric design practioners.

The recommended revisions are based on the findings of this research, and other than a few minor editorial suggestions, do not address topics other than stopping sight distance. Sections of the Green Book potentially affected by the research findings include portions of Chapter II and Chapter III. The remainder of this appendix presents the current text of the Green Book with recommended changes. Deletions of text are shown as ~~strikeouts~~ and additions to the text are shown in **bold**.

The Green Book text presented in this appendix is based on the 1994 edition of the Green Book that, for the first time, incorporates units in the SI or metric system. The revisions to the *Green Book* are shown in Table I-1 at the end of this appendix.

GREEN BOOK CHAPTER II (Design Controls and Criteria)

The following text shows the recommended revisions to the section on speed that appears in pages 61-71 in Chapter II (Design Controls and Criteria) of the 1994 Green Book. The recommended changes are intended primarily to incorporate the research findings.

Design Controls and Criteria 61

Speed

Speed is one of the most important factors to the traveler in selecting alternate routes or transportation modes. The value of a transportation facility in carrying people and goods is judged by its convenience and economy, which are directly related to its speed. The attractiveness of a public transportation system or a new highway are each weighed by the traveler in terms of time, convenience, and money saved. Hence, the desirability of rapid transit may well rest with how rapid it actually is. The speed of vehicles on a road or highway depends, in addition to capabilities of the drivers and their vehicles, upon four general conditions: the physical characteristics of the highway and its roadsides, the weather, the presence of other vehicles, and the speed limitations (either legal or because of control devices). Although any one of these may govern, the effects of these conditions are usually combined.

The objective in design of any engineered facility to be used by the public is to satisfy the demands for service in the safest and most economical manner. The facility should therefore accommodate nearly all demands with reasonable adequacy and also not fail completely under the severe or extreme load. In applying this principle to the design of highways, with particular reference to speed demands, provision should be made for a speed that satisfies nearly all drivers. Only a small percentage of drivers travel at extremely high speeds, and it is not economically feasible to design for them. They can use the highway, of course, but must travel at speeds somewhat less than they consider desirable. On the other hand, the speed chosen for design should not be that used by drivers under unfavorable conditions, such as inclement weather, because the highway then would be unsafe for drivers under favorable conditions, and would not satisfy reasonable demands.

Operating Speed

Operating speed is the ~~highest overall speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions without any time exceeding the safe speed as determined by the design speed on a section-by-section basis.~~ **speed at which drivers are observed operating their vehicles during free flow conditions. The 85th percentile of the distribution of observed speeds is the most frequently used descriptive statistic for the operating speed associated with a particular location or geometric feature.**

Design Speed

Design speed is ~~the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern.~~ **a selected speed used to determine the various geometric design feature of the roadway.** The assumed design speed should be a logical one with respect to the topography, the adjacent land use, and the functional classification of highway. Except for local streets where speed controls are frequently included intentionally, every effort should be made to use as high a design speed as practicable to attain a desired degree of safety, mobility, and efficiency while under the constraints of environmental quality, economics, aesthetics, and social or political impacts. Once selected, all of the pertinent features of the highway should be related to the design speed to obtain a balanced design. Above-minimum design values should be used where feasible, but in view of the numerous constraints often encountered, practical values should be recognized and used. Some features, such as curvature, superelevation, and sight distance, are directly related to, and vary appreciably with, design speed. Other features, such as widths of lanes and shoulders and clearances to walls and rails, are not directly related to design speed, but they affect vehicle speed, and higher standards should be accorded these features for the higher design speeds. Thus, when a change is made in design speed, many design elements of the highway are subject to change.

The design speed chosen should be consistent with the speed a driver is likely to expect. Where a difficult condition is obvious, drivers are more apt to accept lower speed operation than where there is no apparent reason for it. A highway of higher functional classification may justify a higher design speed than a less important facility in similar topography, particularly where the savings in vehicle operation and other operating costs are sufficient to offset the increased costs of right-of-way and construction. A low design speed, however, should not be assumed where the topography is such that drivers are likely to travel at high speeds. Drivers do not adjust their speeds to the importance of the highway, but to their perception of the physical limitations and traffic thereon.

The speed selected for design should fit the travel desires and habits of nearly all drivers. Where traffic and roadway conditions are such that drivers can travel at their chosen speed, there is always a wide range in the speeds at which various individuals operate their vehicles. A cumulative distribution of free flow vehicle speeds has the typical S pattern when plotted as percent of vehicles versus observed speeds. The design speed chosen should be a high-percentile value in this speed distribution curve, i.e., nearly all inclusive of the typically desired speeds of drivers, wherever this is feasible.

The speed distribution curves in Figure II-21 illustrate the range in speed that should be considered in a determination of assumed design speed. A design speed of 110 km/h should be maintained on freeways, expressways, and other major highways. This speed will ensure an adequate design if the speed restriction is removed. Also, a larger percent of the vehicles traveling at the faster speeds will be safely accommodated.

These data lead to the conclusion that where physical features of the highway are the principal speed controls and where most drivers have been conditioned to operate near the speed limit, a top design speed of 120 km/h would fit a very high-percentile speed. On a highway designed for this speed, a small percent of drivers might still operate at higher speed when volume is low and all other conditions are favorable. However, for a design speed of 80 km/h, satisfactory performance could be expected only on certain highways. When the ~~minimum~~ **low** design speeds ~~is~~ **are** used as the criterion, it is important to have the speed limit enforced during off-peak hours.

On many freeways, particularly in suburban and rural areas, a design speed of 100 km/h or higher can be provided with little additional cost above that required for a design speed of 80 km/h. The corridor of the main line may be relatively straight and the character and location of interchanges permit high-speed design. Under these conditions a design speed of 110 km/h is desirable. Flat curvature and ample sight distance usually result in safer highways.

Generally, there is no design speed distinction between a ground-level, an elevated, or a depressed freeway. However, the operating characteristics on elevated freeways differ somewhat from those on depressed freeways. On an elevated highway, traffic exits the facility on downgrade ramps and enters on upgrade ramps. This condition is less desirable than the opposite one on a depressed highway because vehicles, particularly loaded trucks, entering the elevated freeway on an ascending grade require long distances to reach the running speed on the freeway (see section on "Running Speed"). Moreover, vehicles leaving the elevated freeway on a descending grade require additional braking distance to reach the running speed of the arterial street and consequently may tend to slow down on the through traffic lanes in advance of the ramp terminal. Parallel deceleration lanes or longer ramp lengths and lesser grades are frequently used to

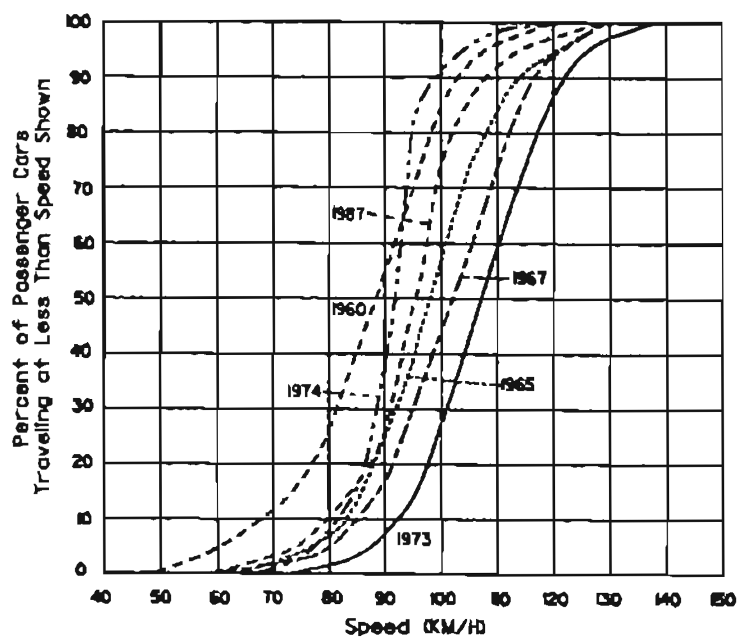


Figure II-21. Distribution of representative passenger car speeds on rural Interstate highways.

reduce the problem of vehicles slowing on the main lanes. Nevertheless, running speeds on elevated freeways are apt to be slightly lower than those on depressed freeways of the same standards, especially when access points are closely spaced. In northern climates, elevated structures are subject to rapid freezing of precipitation as a result of their exposure and may require the use of lesser superelevation rates, which affect both running and design speed. Although speeds on viaducts are less than those on comparable depressed sections, the difference probably is small. Therefore, the appropriate design speeds of 80 to 110 km/h apply to both elevated and depressed freeways.

With an overall range in design speeds of 30 to 120 km/h, it has been found desirable to use increments of 10 km/h. Smaller increments show little distinction in design elements between one design speed and the next higher design speed, and larger increments of 25 to 30 km/h cause too large a difference in design dimensions of features between any two design speeds. In some instances, there may be an advantage in using intermediate increments to effect changes in the design speed. Increments of 10 km/h also may be pertinent to design of turning roadways, ramps, and low-speed roads. The use of 10 km/h increments in design speed below 120 km/h does not preclude the use of smaller increments for traffic control purposes, such as speed-zone signs.

Although the selected design speed establishes the minimum curve radius and minimum sight distance ~~necessary for safe operation~~ **requirements**, there should be no restriction on the use of flatter horizontal curves or greater sight distances where such improvements can be provided as part of economic design. Even in rugged terrain an occasional tangent or flat curve may be desirable. These would not necessarily encourage drivers to speed up; but, if a succession of them is introduced, drivers will naturally resort to higher speeds, and that section of highway should be designed for a higher speed. A substantial length of tangent between sections of curved alinement also is apt to encourage high-speed operation. In such cases a higher speed should be assumed and all geometric features, particularly that of sight distance on crest vertical curves, should be related to it.

A pertinent consideration in selecting design speeds is the average trip length. The longer the trip, the greater the desire for expeditious movement. In design of a substantial length of highway it is desirable where feasible to assume a constant design speed. Changes in terrain and other physical controls may dictate a change in design speed on certain sections. If so, the introduction of a lower design speed should not be done abruptly but should be effected over sufficient distance to permit drivers to change speed gradually before reaching the section of highway with the lower design speed.

Where it is necessary to reduce ~~design speed~~, **horizontal and vertical alignment**, many drivers may not perceive the lower speed condition ahead, and it is important that they be warned well in advance. The changing condition should be indicated by such controls as speed-zone signs and curve-speed signs.

On arterial streets, the design speed control applies to a lesser degree than on other high-type highways. On rural highways or on high-type urban facilities, a certain percentage of vehicles are able to travel at near the ~~safe~~ **design** speed determined by geometric ~~design~~ elements, but on arterial streets the top speeds for several hours of the day are limited or regulated to that at which the recurring peak volumes can be handled. Speeds are governed by the presence of other vehicles traveling en masse both in and across the through lanes and by traffic control devices rather than by the physical characteristics of the street. During periods of low-to-moderate volume, speeds are governed by such factors as speed limits, midblock turns, and intersectional turns, traffic signal spacing and signal timing for progression. When arterial street improvements are being planned, the design speed should be considered with factors such as speed limits, physical and economic constraints, and the likely running speeds that can be attained during offpeak hours, which would influence the selection of the speed design.

Horizontal alinement generally is not the governing factor in restricting speeds on arterial streets. Proposed improvements generally are patterned to the existing street system, and minor horizontal alinement changes are commonly made at intersections. The effect of these alinement changes is usually minor because operation through the intersection is regulated by the type of traffic controls needed to handle the volume of cross and turning traffic. Superelevation may be provided at curves on arterial streets but is developed in a different manner than for open road rural conditions. The wide pavement areas, proximity of adjacent development, control of cross slope and profile for drainage, and the frequency of cross streets and entrances all contribute to the need for lower superelevation rates. Likewise, the width of lanes, offset to curbs, proximity of poles and trees to the traveled way, presence of pedestrians within the right-of-way, and nearness of business or residential buildings, singly and in combination, often nullify speed characteristics of a highway with good alinement and flat profiles. Yet, good alinement and flat profiles should always be strived for in the design of arterial streets, because safety and operating characteristics are improved, particularly during off-peak periods.

Topography can materially affect the choice of design speed on arterial streets. Many of our cities were developed along watercourses and include areas varying from gently rolling to mountainous terrain. The streets originally were constructed with minor grading to fit the topography. Because the arterial street usually is developed to fit an existing street, both through business and residential areas, it generally follows a varying vertical alinement. Once the design speed is determined, the proper sight distance should be assured at all crests. Profile conditions with long, continuous grades should also be designed with proper consideration of speeds of operation of mass transit and commercial vehicles. Extra lanes on the upgrades may be needed so that this portion of the route can match other portions in capacity and enable vehicles that can proceed at reasonable speed to pass slower moving vehicles.

Arterial streets should be designed and control devices regulated, where feasible, to permit ~~running~~ **operating** speeds of 30 to 70 km/h. Lower speeds in this range are applicable for local and collector streets through residential areas and for the arterial streets through the more crowded business areas, while the higher speeds apply to the high-type arterial in the outlying suburban areas. For the arterial streets through the crowded business areas, coordinated signal control through successive intersections generally is necessary to permit even the lower speeds. Many cities have substantial lengths of streets controlled so as to operate at ~~running~~ **operating** speeds of 25 to 40 km/h. At the other extreme in suburban areas, it is common experience on preferred streets to adopt some form of speed zoning or speed control to prevent high operating speeds. In these areas, the infrequent pedestrian or occasional vehicles on a cross street may be unduly exposed to potential accidents from through drivers. Such through drivers gradually gain speed as the frequency of urban restrictions are left behind or such drivers retain their speed of the open road as they enter the city. Thus, although through traffic should be expedited to the extent feasible, it may be equally important to establish a certain speed to reduce potential hazards and to serve local traffic.

A posted speed limit as a matter of practicability, is not the highest speed that might be used by drivers. Instead, it usually approximates the 85-percentile speed value as determined by observing a sizable sample of vehicles. Such a value is within the "pace" or 15 km/h speed range used by most drivers. Speed zones cannot be made to operate properly if arbitrarily determined or selected. In addition, speed zones must be consistent with conditions along the street, the selected cross-section of the street from engineering studies, and must be subject to reasonable enforcement.

With ~~running~~ **operating** speeds of 30 to 70 km/h, it follows that pertinent design speeds for arterial streets and highways would range from 50 to 100 km/h. The selected design speed for an urban arterial highway would depend largely on the spacing of signalized at-grade intersections, the selected type of median cross-section, whether or not curb and gutter is used along the street, and the amount and type of access allowed to the street. As a desirable minimum, elements of a reconstructed urban arterial highway should be designed for a ~~safe~~ operating speed of at least 50 km/h.

The preceding paragraphs describe the basis for, and various considerations that need to be examined when selecting a design speed. From this discussion, it is evident that there are meaningful differences between the design criteria applicable to low- and high-speed designs. Because of these distinct differences, it is desirable to establish certain limits. For application in this publication, the upper limit for low or lower design speed usually is 60 km/h, and the minimum limit for high speed design is 80 km/h. The intermediate design speed of 70 km/h could be considered as either low speed or high speed depending upon the specific conditions along the street, and such conditions would govern in the selection of appropriate design criteria.

Running Speed

In design it is necessary to know actual vehicle speeds for traffic en masse to be expected on highways of different design speeds and various volume conditions. Speed of operation is one measure of the service that a highway renders, and it affords a means of evaluating road-user cost and benefits. The running speed is the speed of a vehicle over a specified section of highway, being the distance traveled divided by the running time (the time the vehicle is in motion).

One means of obtaining an equivalent average running speed on an existing facility where flow is reasonably continuous is to measure the spot speed. The average spot speed is the arithmetic mean of the speeds of all traffic at a specified point. For short sections of highway on which speed characteristics do not vary materially, the average spot speed may be considered as being representative of the average running speed. On longer stretches of rural highway, spot speeds measured at several points, where each represents the speed characteristics pertinent to a selected segment of highway, may be averaged (taking relative lengths into account) to represent the average running speed.

Average spot speeds, which generally are indicative of average running speeds, have been measured over a period of years in many States on highway sections of favorable alinement. The average speed slowly increased over the years, then leveled out, and subsequently dropped with the advent of the 55-mph speed limit. Since then it has decreased very slightly on highways where the 55-mph speed limit is still in effect. On interstate highways with 105 km/h speed limits, the increase has been greater.

Experience on horizontal curves shows that speeds are lower than those on tangent alinement and that the difference between average spot speed and ~~calculated~~ **inferred** design speed on such curves becomes less as the radius of curvature decreases. In this regard, it is generally accepted that a greater proportion of drivers operate near or at the design speed on highways with low design speed than on highways with high design speed. It is also known that some sections of low design speed highways are frequently overdriven, with an appreciable number of drivers exceeding the **inferred** design speed.

Observed speeds of free-moving vehicles on horizontal curves indicate that low design speed curves yield an average spot speed close to the design speed; on high design speed curves the average speed is substantially below the design speed and approaches the average spot speed found on long stretches of tangent alinement. Because horizontal curvature is the principal factor related to design speed of rural highways and since average spot speed approximates the average running speed for such conditions, a useful relation between the highway design speed and the average running speed (for low-volume conditions) may be established from these data. Comparing the observed average speeds with ~~calculated~~ **inferred** design speeds, it is found that on sections having a 50 km/h design speed, the average running speed is approximately 90 to 95 percent of the design speed.

The general relation between design speed and average running speed (the average for all traffic or component of traffic, being the summation of distances divided by the summation of running times; it is approximately equal to the average of the running speeds of all vehicles being considered) is illustrated in Figure II-22. The upper curve represents the conditions for low traffic volume as just described. As traffic volume increases on any highway, the average running speed decreases because of interference among vehicles. The curve labeled "Intermediate Volume" represents the relation between design speed and average running speed when the volume approximates the design service volume for rural highways. Should the volume exceed the intermediate level, the average running speed would be further lowered, and in the extreme case, where the volume is approaching the capacity of the highway, the speed of traffic is influenced more by congestion than by the design speed, especially where the design speed is above 80 km/h. The relation between design speed and average running speed for very high traffic volumes is illustrated by the lower cure in Figure II-22. This curve ~~is of academic interest only. It~~ establishes a limiting condition for average running speeds but it is of little value in design. Highways should usually be designed to accommodate their traffic volumes without being subjected to the high degree of congestion represented by this curve.

A design that satisfies the requirements for average running speed at low volume is adequate for traffic using the highway when the volumes are higher and the speeds are lower. At low volumes about 50 percent of all vehicles travel at speeds within 10 km/h of the average running speeds, as shown by the speed distribution curves in Figure II-21. For volumes in the intermediate range about 90 percent of all vehicles travel at or less than the average running speed representative of low volumes. For this reason, low volumes control certain highway elements, such as lane and shoulder widths, treatment of intersections curves, and speed-change lanes.

Average running speed on a given highway varies somewhat during the day, depending primarily on the volume of traffic. Therefore, when reference is made to running speed it should be clear whether this speed is for peak hours or offpeak hours or whether is an average for the day. The first two are of concern in design and operation; the latter is of importance in economic analyses.

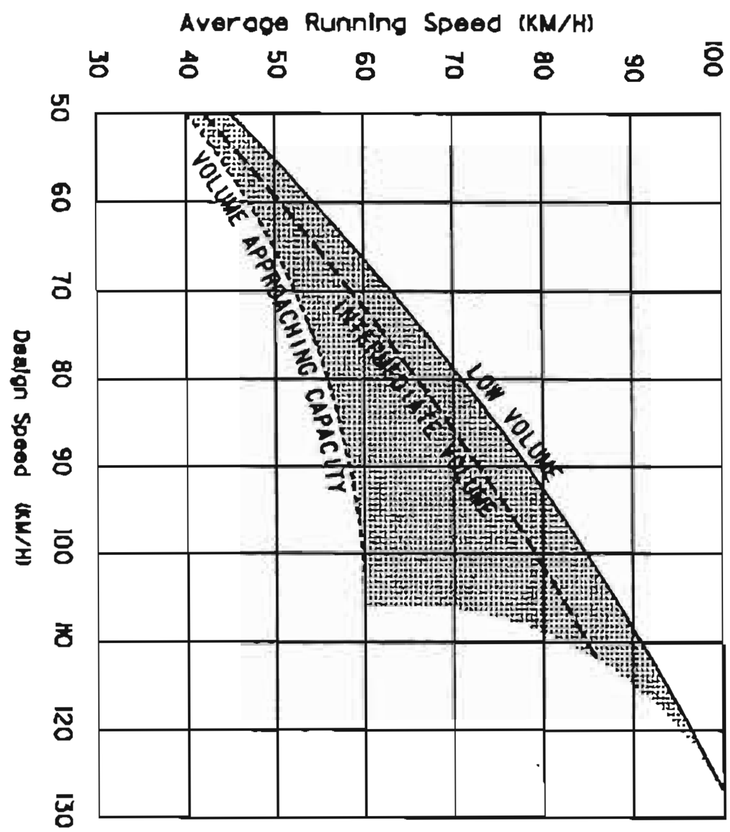


Figure II-22. Relation of average running speed and volume conditions.

Figure II-23 depicts the relationship between average speed of an ideal traffic stream and ideal flow rate for a 15-minute period. Figure II-23 depicts two important characteristics:

1. There is a substantial range of flow over which speed is relatively insensitive to flow: this range extends to fairly high flow rates.
2. As flow approaches capacity, speed drops off at a sharp rate.

The data for Figure II-23 are taken from the *Highway Capacity Manual* (HCM) (11).

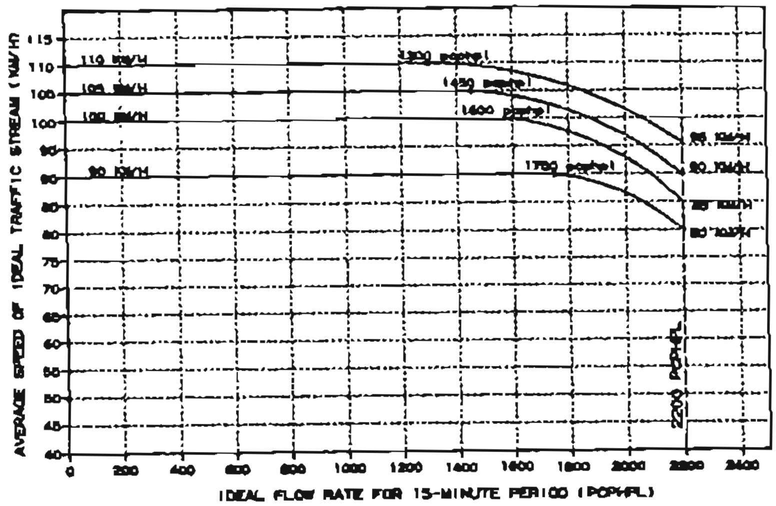


Figure II-23. Speed flow characteristics for basic freeway sections (for ideal conditions) (11).

GREENBOOK CHAPTER III (Elements of Design)

The following text shows the recommended revisions to the sections on sight distance (pages 117-138), sight distance on horizontal curves (pages 219-223), and vertical curves (pages 279-293) that appear in Chapter II of the 1994 Green Book. The recommended changes are intended primarily to incorporate the research findings.

Chapter III
ELEMENTS OF DESIGN

SIGHT DISTANCE

General Considerations

The ability to see ahead is of the utmost importance in the safe and efficient operation of a vehicle on a highway. On a railroad, trains are confined to a fixed path, yet a block signal system and trained operators are necessary for safe operation. On the other hand, the path and speed of motor vehicles on highways and streets are subject to the control of drivers whose ability, training, and experience are quite varied. For safety on highways the designer must provide sight distance of sufficient length that drivers can control the operation of their vehicles to avoid striking an unexpected object on the traveled way. Certain two-lane highways should also have sufficient sight distance to enable drivers to occupy the opposing traffic lane for passing overtaken vehicles without risk of accident. Two-lane rural highways should generally provide such passing sight distance at frequent intervals and for substantial portions of their length. Conversely, it normally is of little practical value to provide passing sight distance on two-lane urban streets or arterials. The length and interval of passing sight distance should be compatible with the criteria established in the chapter pertaining to that specific highway or street classification.

Sight distance is discussed in four steps: (1) the distances required for stopping, applicable on all highways; (2) the distances required for the passing of overtaken vehicles, applicable only on two-lane highways; (3) the distances needed for decisions at complex locations; and (4) the criteria for measuring these distances for use in design. The design of alinement and profile to provide these distances and to meet these criteria are described later in this chapter. The special conditions related to sight distances at intersections are discussed in Chapter IX.

Stopping Sight Distance

Sight distance is the length of roadway ahead visible to the driver. The minimum sight distance available on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Although the greater length is desirable, sight distance at every point along the highway should be at least that required for a below-average operator or vehicle to stop in this distance.

Stopping sight distance is the sum of two distances: the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied and the distance required to stop the vehicle from the instant brake application begins. These are referred to as brake reaction distance and braking distance, respectively.

Brake Reaction Time

Brake reaction time is 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. This interval includes the time required to make the decision that a stop is necessary. Under certain conditions, such as emergency conditions denoted by flares or flashing lights, operators accomplish these tasks almost instantly. Under most other conditions the operator must subconsciously associate the object ahead with stationary objects adjacent to the roadway, such as walls, fences, trees, poles, or bridges, to determine that the object is also stationary or moving at a slow speed. These determinations take time, the amount of which varies considerably depending on the distance to the object, the acuity of the operator, the natural rapidity with which the driver reacts, atmospheric visibility, the type and the condition of the roadway, and the type, color, and condition of the hazard. Vehicle speed and the roadway environment probably also influence reaction time. Normally, an operator traveling at or near the design speed is more alert than one traveling at a lesser speed. An operator on an urban facility confronted by innumerable possibilities for conflicts from parked vehicles, driveways, and cross streets is also likely to be more alert than the same operator on a limited-access facility where such conditions should be almost nonexistent.

The study (1) referred to in Chapter II was based on data from 321 drivers who expected to apply their brakes. The median reaction-time value for these drivers was 0.66 s with 10 percent requiring 1.5 s or longer. These findings correlate with those of earlier studies in which alerted drivers were also used. Another study (2) gives 0.64 s as the average value; 5 percent of the drivers required over 1 s. In a third study (3) reaction-time values ranged from 0.4 to 1.7 s. In the Johansson and Rumar study (1), when the signal was unexpected, the drivers' responses were found to increase by approximately 1 s or longer; some reaction times being 1.5 s or more. This increase substantiated earlier laboratory and road tests in which the conclusion was drawn that the driver who required 0.2 to 0.3 s under alert conditions required 1.5 s under normal conditions.

Minimum reaction times thus could be at least 1.64 s; 0.64 s for alerted drivers plus 1 s for the unexpected signal. Because the studies used simple prearranged signals, they represent the least complex of roadway conditions. Even under these simple conditions it was found that some operators may take over 3.5 s to respond. Because actual conditions on the highway are generally more complex than those of the studies and because there is wide diversity in the reaction times required, it is evident that the value adopted should be greater than 1.64 s. In determination of sight distance for design, the reaction time ~~should be larger than the average for all drivers under normal conditions. It~~ should be large enough to include the reaction time required for nearly all drivers under most highway conditions. For approximately 90 percent of the drivers in the first study mentioned, a reaction time of 2.5 s was found to be adequate. A reaction time of 2.5 s has thus been assumed in the development of Table III-1.

A reaction time of 2.5 s is considered adequate for more complex conditions than those of the various studies, but it is not adequate for the most complex conditions encountered by the driver. Additional consideration of the most complex conditions such as those found at multiphase at-grade intersections and ramp termini at through roadways can be found later in this chapter in the section "Decision Sight Distance."

Braking Distance

The approximate braking distance of a vehicle on a level roadway may be determined by the use of the standard formula:

$$d = \frac{V^2}{254f}$$

$$\boldsymbol{d = 0.039V^2/a}$$

where: d = braking distance, m;
V = initial speed, km/h; and
~~f = coefficient of friction between tires and roadway.~~
a = driver deceleration, m/s²

~~In this formula for braking distance the f factor is used as an overall or a single value that is representative for the whole of the speed change. Measurements show that f is not the same for all speeds. It decreases as the initial speed increases. It varies considerably because of many physical elements such as air pressure of tires, composition of tires, tire tread pattern and depth of tread, type and condition of the pavement surface, and the presence of moisture, mud, snow or ice.~~

~~Design Speed (km/h)~~	~~Assumed Speed for Condition (km/h)~~	~~Brake Reaction Time (s)~~	~~Brake Reaction Distance (m)~~	~~Coefficient of Friction[a] f~~	~~Braking Distance on Level (m)~~	~~Stopping Sight Distance for Design (m)~~
~~30~~	~~30-30~~	~~2.5~~	~~20.8-20.8~~	~~0.40~~	~~8.8-8.8~~	~~29.6-29.6~~
~~40~~	~~40-40~~	~~2.5~~	~~27.8-27.8~~	~~0.38~~	~~16.6-16.6~~	~~44.4-44.4~~
~~50~~	~~47-50~~	~~2.5~~	~~32.6-34.7~~	~~0.35~~	~~24.8-28.1~~	~~57.4-62.8~~
~~60~~	~~55-60~~	~~2.5~~	~~38.2-41.7~~	~~0.33~~	~~36.1-42.9~~	~~74.3-84.6~~
~~70~~	~~63-70~~	~~2.5~~	~~43.7-48.6~~	~~0.31~~	~~50.4-62.2~~	~~94.1-110.8~~
~~80~~	~~70-80~~	~~2.5~~	~~48.6-55.5~~	~~0.30~~	~~64.2-83.9~~	~~112.8-139.4~~
~~90~~	~~77-90~~	~~2.5~~	~~53.5-62.5~~	~~0.30~~	~~77.7-106.2~~	~~131.2-168.7~~
~~100~~	~~85-100~~	~~2.5~~	~~59.0-69.4~~	~~0.29~~	~~98.0-135.6~~	~~157.0-205.0~~
~~110~~	~~91-110~~	~~2.5~~	~~63.2-76.4~~	~~0.28~~	~~116.3-170.0~~	~~179.5-246.4~~
~~120~~	~~98-120~~	~~2.5~~	~~68.0-83.3~~	~~0.28~~	~~134.9-202.3~~	~~202.9-285.6~~

[a] ~~Values of coefficient of friction generally approximate curves 9 and 10 (coefficient of friction for wet PC concrete and wet~~

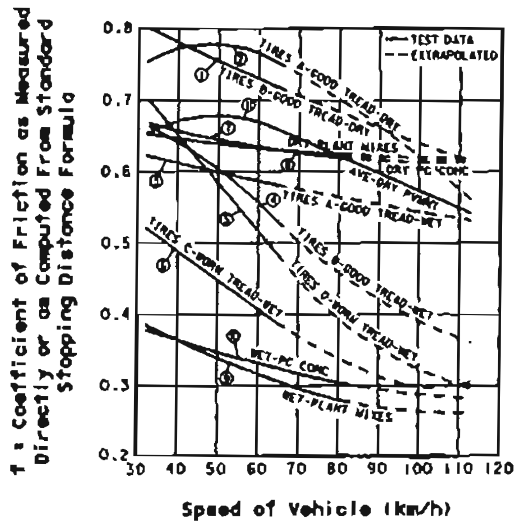
~~Table III-1. Stopping sight distance (wet pavements).~~

~~The braking distance also depends on the braking system of the vehicle. The several variables are allowed for if f is computed for each test from the standard formula. It thus represents the equivalent constant friction factor. The values of f in Figure III-1 (A and B) were calculated on this basis for some of the curves that represent tests in which only speed and distance were recorded.~~

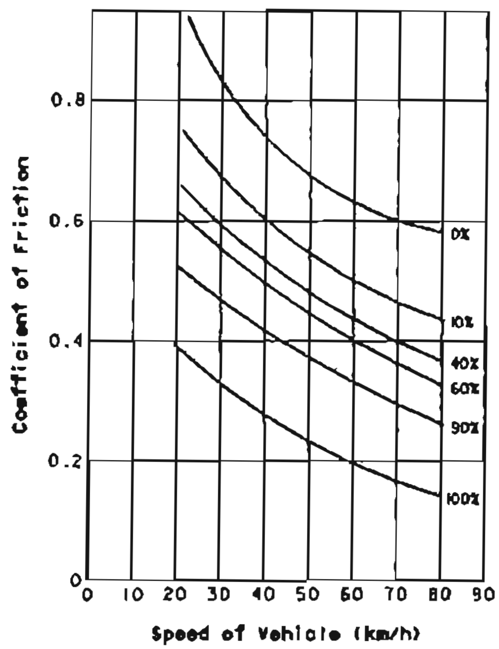
~~Figure III-1 A illustrates friction coefficients found by different investigators. Curves 1 to 6 are from a study (4) in which more than 1,000 measurements of forward skidding were made on 32 pavements in both wet and dry conditions. Several types of tires were being used. Coefficients of friction were computed by using actual stopping distances in the standard stopping formula. Curves 7 and 8 are representative of several curves of a study (5) in which over 50 surfaces were tested in the dry condition by three different methods using three types of tires. Curves 9 and 10 from the same study are representative of wet pavements. Curve 11 is the calculated equivalent f value for stopping sight distances measured (3) on high-type pavement; these were the only tests that included stops from speeds of about 100 to 110 km/h. This curve is the average of all stops measured, but comparison with on-the-road samples (at low speeds) shows that the vehicles and drivers were somewhat better than the average on the highways.~~

~~Because of the lower coefficients of friction on wet pavements as compared with dry, the wet condition governs in determining stopping distances for use in design. The coefficients of friction used for design criteria should represent not only wet pavements in good condition but also surfaces approaching the end of their useful lives. The values should encompass nearly all significant pavement surface types and the likely field conditions. Figure III-1B summarizes the results of a series of 600 measurements made on modern pavements in Germany (6). Each road section was tested at 20, 40, 60, and 80 km/h by a locked-wheel trailer. Because the test methods differ, there is no direct correlation between these curves and those of Figure III-1A or Table III-1. However, Figure III-1B does indicate that there is a wide variation between the coefficients of friction for various pavements, which reflects the effect that surface texture has on stopping sight distance and, therefore, the importance of this factor.~~

~~The friction factor values used in calculating safe stopping distances should allow for worn tires as well as for new tires and for nearly all types of treads and tire compositions. The friction factor values used should also encompass the differences in vehicle and driver braking from various speeds. On the other hand, the values used need not be so low as to be suitable for pavements under icy conditions. Preferably, the f values used for design should be nearly all-inclusive, rather than average; however, available data are not fully detailed over the range for all these variables, and conclusions must be made in terms of the safest reported average values. The coefficients of friction in Table III-1 have been selected based on these criteria as being appropriate for the calculation of stopping sight distances.~~



A. Skid resistance for various tire and pavement conditions.



B. Skid Resistance of Pavement - Percent of Pavements Which Exceed Coefficient of Friction.

Figure III-1. Variation in coefficient of friction with vehicular speed.

~~Comparison with Figure III-1A shows that the range of values in Table III-1 is generally conservative; the upper limit reflects the concern for the more recent measurements such as those of Figure III-1B, which show a few coefficients for wet pavements with values near or less than the values used for developing the lower limits for stopping sight distance.~~

~~The average running speed for low-volume conditions rather than design speed is used in formulating the limiting values for minimum stopping distance. This speed is the initial value given in the second column of Table III-1. Studies show that many operators drive just as fast on wet pavements as they do on dry. To account for this factor, design speed in place of average running speed is used to formulate stopping distance values, as shown by the higher values in the second column of Table III-1.~~

Studies documented in the literature show that most drivers choose decelerations greater than 4.5 m/s^2 when confronted with the need to stop for an unexpected object in the roadway. Approximately 90 percent of all drivers on wet surfaces choose decelerations that are greater than 3.4 m/s^2. These decelerations are within the driver's capability to stay within their lane and maintain steering control during the braking maneuver on wet surfaces (R).

Thus, 3.4 m/s^2 (a comfortable deceleration for most drivers) is recommended as the deceleration threshold for determining required stopping sight distance. Implicit in this deceleration threshold is the requirement that the vehicle braking system and pavement friction values are at least equivalent to 3.4 m/s^2 (0.34 g). Skid data show that most wet pavement surfaces on state maintained roadways exceed this threshold (R). In addition, braking data show that most vehicle braking systems can exceed the skidding friction values for the pavement (R).

Design Values

The sum of the distance traversed during the brake reaction time and the distance to stop the vehicle is the minimum stopping sight distance. The computed distances for wet pavements and for various speeds at the assumed conditions are shown in Table III-1 and were developed using the following formula:

$$d = (0.278)(t)(V) + \frac{V^2}{254f} + 0.039V^2/a$$

where: t = brake reaction time, generally assumed to be 2.5 s;
V = initial speed, km/h; and
~~f = coefficient of friction between tires and roadway.~~
a = driver decelerations m/s^2

Continued -

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~~Any length of stopping sight distance within the range of values established in Table III-1 is acceptable for a specific speed. However, values approaching or~~ **Values** exceeding the **recommended stopping sight distances** ~~upper limit of the range~~ should be used as the basis for design wherever conditions permit. Use of the ~~upper limit~~ **larger** values increases the margin of safety. ~~by providing for the drivers who operate at or near the design speed during wet weather.~~ To ensure that new pavements will have initially, and will retain, coefficients of friction comparable to ~~those~~ **the deceleration values** given in the table, designs should meet the criteria established in the AASHTO *Guidelines for Skid Resistant Pavement Design* (7). ~~Although research data may demonstrate that proposed pavements will have initially, and will retain coefficients of friction greater than those given in Table III-1, this finding should not be considered justification for using stopping sight distance values less than the minimums contained in this table. Variations in results (particularly because of the methods of determining coefficients of friction) preclude such direct comparisons. If portions of existing~~

~~pavements are to be retained, skid tests should be made. Although direct comparisons cannot be made, the need for increased lengths of stopping sight distance should be considered when tests yield coefficients of friction significantly below those in Table III-1.~~

~~Table III-1 includes data for the higher design speeds for wet conditions where no actual test data are available. Values for design speeds above 100 km/h are extrapolated from the curves for lower speeds in Figure III-1A.~~

Effect of Grade on Stopping

When a highway is on a grade, the standard formula for braking distance is the following:

$$d = \frac{V^2}{254 \quad (f \pm G) \quad (a/9.81) \pm G)}$$

in which G is the percent grade divided by 100, and the other terms are as previously stated. The stopping distances on upgrades are shorter; those on downgrades are longer. The stopping sight distances on various grades are indicated in Table III-2. These corrections are computed for wet conditions, the assumed design criterion used in Table III-1. The brake reaction time is assumed to be the same as for level conditions. Design speed is used in calculating ~~downgrade~~ corrections. ~~average running speed in calculating upgrade corrections. The different criteria for descending and ascending grades are based on the effect grades have on the speed of individual vehicles, particularly trucks; the effect these vehicles have on the overall speed of the traffic stream; and the premise that many drivers, particularly those in automobiles, do not compensate completely for the changes in speed caused by grades.~~

On nearly all roads and streets the grade is traversed by traffic in both directions, but the sight distance at any point on the highway generally is different in each direction, particularly on straight roads in rolling terrain. As a general rule, the sight distance available on downgrades is larger than on upgrades, more or less automatically providing the necessary corrections for grade. This may explain why some design offices do not make corrections in stopping sight distance because of grade. Exceptions are one-way roads or streets, as on divided highways with independent design profiles for the two roadways. For these the separate grade corrections are in order and the refinement in design is in keeping with the overall standards used.

Continued

Table III-1. Recommended Stopping Sight Distances for Design.

Initial Speed (km/h)	Perception-Brake Reaction Time (s)	Perception-Brake Reaction Distance (m)	Deceleration (m/s^2)	Braking Distance (m)	Stopping Sight Distance for Design (m)
30	2.5	20.8	3.4	10.2	31.0
40	2.5	27.8	3.4	18.2	45.9
50	2.5	34.7	3.4	28.4	63.1
60	2.5	41.7	3.4	40.8	8.2
70	2.5	48.6	3.4	55.6	104.2
80	2.5	55.6	3.4	72.6	128.2
90	2.5	62.5	3.4	91.9	154.4
100	2.5	69.4	3.4	113.5	182.9
110	2.5	76.4	3.4	137.3	213.7
120	2.5	83.3	3.4	163.4	246.7

Note: Shading represents sight distances that are beyond most driver's visual capabilities for detecting small and/or low contrast objects.

~~Design Speed km/h~~	~~Stopping Sight Distance (m) for Downgrades~~			~~Assumed Speed for Condition (km/h)~~	~~Stopping Sight Distance (m) for Upgraded~~		
	~~3%~~	~~6%~~	~~9%~~		~~3%~~	~~6%~~	~~9%~~
~~30~~	~~30.4~~	~~31.2~~	~~32.2~~	~~30~~	~~29.0~~	~~28.5~~	~~28.0~~
~~40~~	~~45.7~~	~~47.5~~	~~49.5~~	~~40~~	~~43.2~~	~~42.1~~	~~41.2~~
~~50~~	~~65.5~~	~~68.6~~	~~72.6~~	~~47~~	~~55.5~~	~~53.8~~	~~52.4~~
~~60~~	~~88.9~~	~~94.2~~	~~100.8~~	~~55~~	~~71.3~~	~~68.7~~	~~66.6~~
~~70~~	~~117.5~~	~~125.8~~	~~136.3~~	~~63~~	~~89.7~~	~~85.9~~	~~82.8~~
~~80~~	~~148.8~~	~~160.5~~	~~175.5~~	~~70~~	~~107.1~~	~~102.2~~	~~98.1~~
~~90~~	~~180.6~~	~~195.4~~	~~214.4~~	~~77~~	~~124.2~~	~~118.8~~	~~113.4~~
~~100~~	~~220.8~~	~~240.6~~	~~256.9~~	~~85~~	~~147.9~~	~~140.3~~	~~133.9~~
~~110~~	~~267.0~~	~~292.9~~	~~327.1~~	~~91~~	~~168.4~~	~~159.1~~	~~151.3~~
~~120~~	~~310.1~~	~~341.0~~	~~381.7~~	~~98~~	~~190.0~~	~~179.2~~	~~170.2~~

~~**Table III-2. Effect of grade on stopping sight distance – wet conditions.**~~

NOTE: TABLE III-2 TO BE REDONE.

Variation for Trucks

The ~~**derived**~~ **recommended** minimum stopping sight distances directly reflect passenger car operation and might be questioned for use in design for truck operation. Trucks as a whole, especially the larger and heavier units, require longer stopping distances from a given speed than passenger vehicles do. However, there is one factor that tends to balance the additional braking lengths for trucks for given speeds with those for passenger cars. The truck operator is able to see the vertical features of the obstruction substantially farther because of the higher position of the seat of the vehicle. Separate stopping sight distances for trucks and passenger cars, therefore, are not used in highway design standards.

There is one situation that should be treated with caution, in which every effort should be made to provide stopping sight distances greater than the minimum design value. When horizontal sight restrictions occur on downgrades, particularly at the ends of long downgrades, the greater height of eye of the truck operator is of little value, even when the horizontal sight obstruction is a cut slope, when (on long downgrades) truck speeds may closely approach or exceed those of passenger cars. Although the average truck operator tends to be more experienced than the average passenger car operator and quicker to recognize hazards, it is best under such conditions to supply a stopping sight distance that meets or exceeds the values in Table III-1.

Criteria for Measuring Sight Distance

Sight distance is the distance along a roadway that an object of specified height is continuously visible to the driver. This distance is dependent on the height of the driver's eye above the road surface, the specified object height above the road surface, and the height of sight obstructions within the line of sight.

Height of Driver's Eye

For sight distance calculations for passenger vehicles, the height of the driver's eye is considered to be ~~1070~~ **1080** mm above the road surface. This value is based on studies ~~(14, 15, 16, 17)~~ **(14)** which show that average vehicle heights decreased since 1960 to 1300 mm with a comparable decrease in average eye heights to ~~1070~~ **1080** mm. ~~The average vehicle heights decreased 66 mm in this period, which correlates well with the 53 mm reduction in average eye heights. In the same time period the minimum height of eye decreased 64 mm to 1000 mm. Because of this significant change in the minimum eye heights, the design eye height has been reduced from 1140 to 1070 mm.~~ This change in eye height has the effect of lengthening minimum crest vertical curves by approximately 5 percent, thereby providing about 2.5 percent more sight distance. Because of various factors that appear to place practical limits on any further decreases in passenger car heights and the relatively small increases that further change would mandate in lengths of vertical curves, ~~1070~~ **1080** mm is considered to be the height of driver's eye for measuring both stopping and passing sight distances. **This value encompasses 90 percent of all passenger car driver eye heights.** For large trucks the driver eye height ranges from ~~1.8 to 2.4 m~~, **2.3 to 2.6**, the most common being 2.4 m. For design 2.4 m is the assumed eye height for trucks.

Height of Object

For stopping sight distance calculations, the height of object is considered to be ~~150~~ **600** mm above the road surface. For passing sight distance calculations, the height of object is considered to be 1300 mm above the road surface.

Stopping sight distance object. ~~The object height of 150 mm was adopted for stopping sight distance calculation purposes in 1965~~. The basis for ~~its~~ selection **of a 600 mm** object height was ~~largely~~ ~~an arbitrary~~ rationalization of possible hazardous object size and a driver's ability to perceive and react to a hazardous situation. ~~If other vehicles were the only likely hazard to be encountered, the height of vehicle taillights, 460 mm to 600 mm, would be sufficient object height. Such a height, however, would preclude a driver's seeing small animals, rocks, or other debris that are likely to be encountered in the roadway~~. It is considered that a ~~150~~ **600** mm-high object is representative of the ~~lowest~~ **realistic** object that can create a hazardous condition and be perceived as a hazard by a driver in time to stop before reaching it. Using object heights of less than ~~150~~ **600** mm for stopping sight distance calculations results in ~~considerably~~ longer crest vertical curves **without documented safety benefits.** ~~For example, if the roadway surface is used as the sighted object, crest vertical curves would have to be about 85 percent longer than when 150 mm is used as the object height.~~ The object height of less than ~~150~~ **600** mm could ~~substantially~~ increase construction costs because additional excavation would be required to provide the longer crest vertical curves. It is also doubtful that the driver's ability to perceive a hazardous situation would be increased **because of limitations of the driver's visual capabilities.**

Passing sight distance object. The object height of 1300 mm is adopted for passing sight distance calculations, superseding the 1400-mm object height, which had been used since 1940. Because vehicles are the objects that must be seen when passing and because the height of the average passenger vehicle body has been reduced to its current 1300-mm height above the pavement, this height will be used for calculation purposes. Passing sight distances calculated on this basis are also considered adequate for night conditions because the beams of the headlights of an opposing vehicle generally are seen from a greater distance than its top could be seen in the daytime.

Sight Obstructions

On tangents the obstruction that limits the driver's sight distance is the road surface at some point on a crest vertical curve. On horizontal curves the obstruction that limits the driver's sight distance may be the road surface at some point on a crest vertical curve, or it may be some physical feature outside of the traveled way, such as a longitudinal barrier, a bridge-approach fill slope, a tree, foliage, or the backslope of a cut section. Accordingly, all highway construction plans should be checked in both the vertical and horizontal plane for sight distance obstructions.

Measuring and Recording Sight Distance on Plans

The design of horizontal alinement and vertical profile using sight distance and other criteria is covered later in this chapter, particularly the detail design of horizontal and vertical curves. Sight distance, however, should be considered in the preliminary stages of design when both the horizontal and vertical alinement are still subject to adjustment. By determining graphically the sight distances on the plans and recording them at frequent intervals, the designer can appraise the overall layout and effect a more balanced design by minor adjustments in the plan or profile. Methods for scaling sight distances are demonstrated in Figure III-3. The figure also shows a typical sight distance record that would be shown on the final plans.

Because the view of the highway ahead may change rapidly in a short distance, it is desirable to measure and record sight distance for both directions of travel at each station. Both horizontal and vertical sight distances should be measured and the shorter lengths recorded. In the case of two-lane highways, passing sight distance in addition to stopping sight distance should be measured and recorded.

Sight distance charts such as those in Figures III-39 through III-42 may be used to establish minimum lengths of vertical curves. Charts similar to Figures III-24A and III-24B are useful for determining the radius of horizontal curve or the lateral offset therefrom needed to provide the required sight distance. Once the horizontal and vertical alinements are tentatively established, the practical means of examining sight distances along the proposed highway is by direct scaling on the plans.

Horizontal sight distance on the inside of a curve is limited by obstructions such as buildings, hedges, wooded areas, highground, or other topographic features. These generally are plotted on the plans. Horizontal sight is measured with a straightedge, as indicated at the upper left in Figure III-3. The cut slope obstruction is shown on the worksheets by a line representing the proposed excavation slope at a point ~~600~~ **840** mm (approximate average of ~~1070~~ **1080** mm and ~~150~~ **600** mm) above the road surface for stopping sight distance and at a point about 1100 mm above the road surface for passing sight distance. The position of this line with respect to the centerline may be scaled from the plotted highway cross sections. Preferably, the stopping sight distance should be measured between points on the one traffic lane, and passing sight distance from the middle of one lane to the middle of the other lane. Such refinement on two-lane highways generally is not necessary and measurement to the centerline or traveled way edge is suitable. Where there are changes of grade coincident with horizontal curves that have sight-limiting cut slopes on the inside, the line-of-sight intercepts the slope at a level either lower or higher than the assumed average height. In measuring sight distance the error in the use of the assumed 600- or 1100-mm height usually can be ignored.

Vertical sight distance may be scaled from a plotted profile by the method illustrated at the right center of Figure III-3. A transparent strip with parallel edges 1300 mm apart and with scratched lines ~~150~~ **600** mm and ~~1070~~ **1080** mm from the upper edge, in accordance with the vertical scale, is a useful tool. The ~~1070~~-**1080** mm line is placed on the station from which the vertical sight distance is desired, and the strip is pivoted about this point until the upper edge is tangent to the profile. The distance between the initial station and the station on the profile intersected by the ~~150~~ **600** mm line is the stopping sight distance. The distance between the initial station and the station on the profile intersected by the lower edge of the strip is the passing sight distance.

A simple sight distance record is shown in the lower part of Figure III-3. Sight distances in both directions are indicated by arrows and figures at each station on the plan and profile sheet of the proposed highway. To avoid the extra work of measuring the unusually long sight distances that may occasionally be found, a selected maximum value may be recorded. In the example shown, all sight distances of more than 1000 m are recorded as 1000+, and where this occurs for several consecutive stations, the intermediate values are omitted. Sight distances less than 500 m may be scaled to the nearest 10 m and those greater than 500 m to the nearest 50 m. The available sight distances along a proposed highway also may be shown by other methods. Several States use a sight distance graph, plotted in conjunction with the plan and profile of the highway, as a means of demonstrating sight distances. Sight distances can easily be determined also where plans and profiles are drawn using computer aided design and drafting systems (CADD).

Sight distance records for two-lane highways may be used to advantage to tentatively determine the marking of no-passing zones in accordance with criteria given in the MUTCD (8). Marking of such zones is an operational rather than a design problem. No-passing zones thus established serve as a guide for markings when the highway is completed; the zones so determined should be checked and adjusted by field measurements before actual markings are placed.

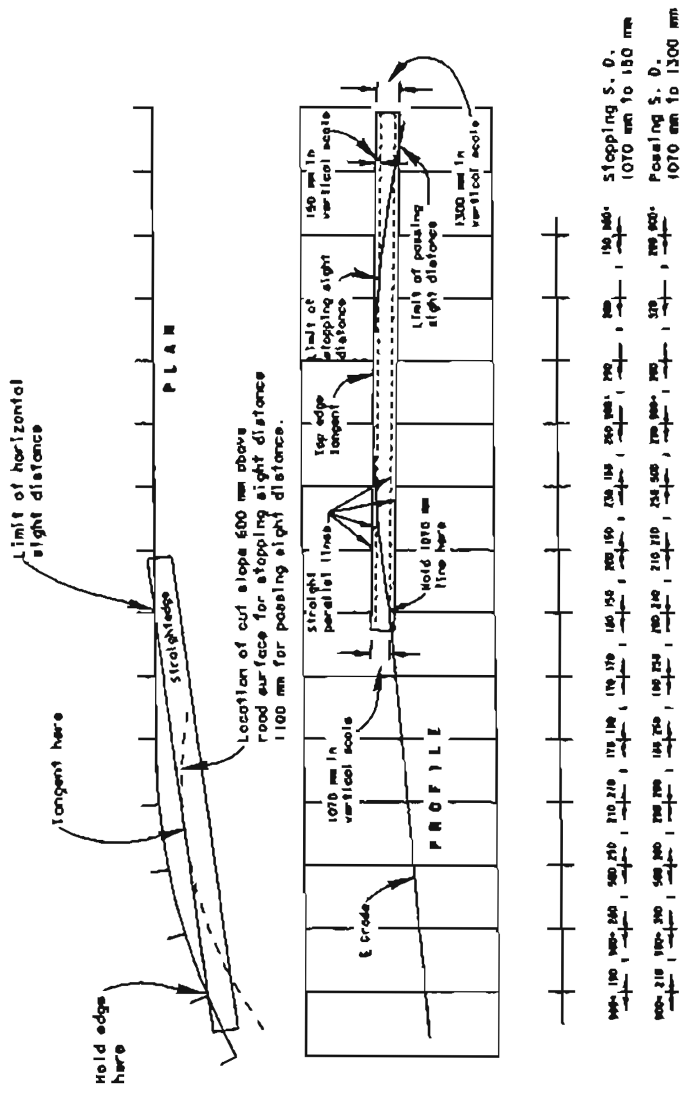


Figure III-3. Scaling and recording sight distances on plans.

Sight distance records also are useful on two-lane highways for determining the percentage of length of highway on which sight distance is restricted to less than the passing minimum, which is important in evaluating capacity. With recorded sight distances, as in the lower part of Figure III-3, it is a simple process to determine the percentage of length of highway with a given sight distance or greater.

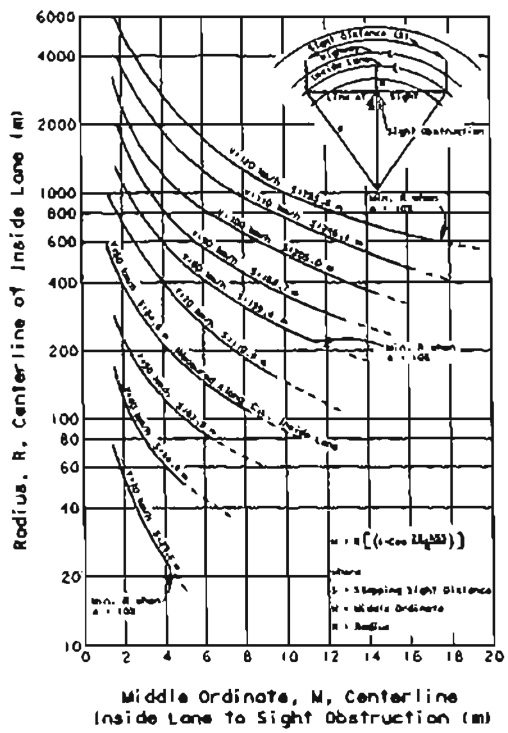
Sight Distance on Horizontal Curves

Another element of horizontal alinement is the sight distance across the inside of curves. Where there are sight obstructions (such as walls, cut slopes, buildings, and longitudinal barriers) on the inside of curves, a design to provide adequate sight distance may require adjustment in the normal highway cross section or change in alinement if the obstruction cannot be removed. Because of the many variables in alinement and cross sections and in number, type, and location of possible obstructions, specific study usually is necessary for each condition. Using design speed and a selected sight distance as a control, the designer should check the actual condition and make the necessary adjustments in the manner most fitting to provide adequate sight distance.

Stopping Sight Distance

For general use in design of a horizontal curve, the sight line is a chord of the curve, and the applicable stopping sight distance is measured along the centerline of the inside lane around the curve. Figure III-24~~A and III-24B~~ is a ~~are~~ design charts showing the required middle ordinates for clear sight areas to satisfy the ~~upper and lower values, respectively~~, of stopping sight distance required for curves of various radii.

These design chart utilize the stopping sight distance values of Table III-1. ~~The overlap of the ranges at the higher design speeds in this table precludes development of a single design chart. However, use of the two charts in combination provides the same result; namely, a value that exceeds the minimum~~



~~**Figure III-24A. Range of upper values - relation between radius and value of middle ordinate necessary to provide stopping sight distance on horizontal curves.**~~

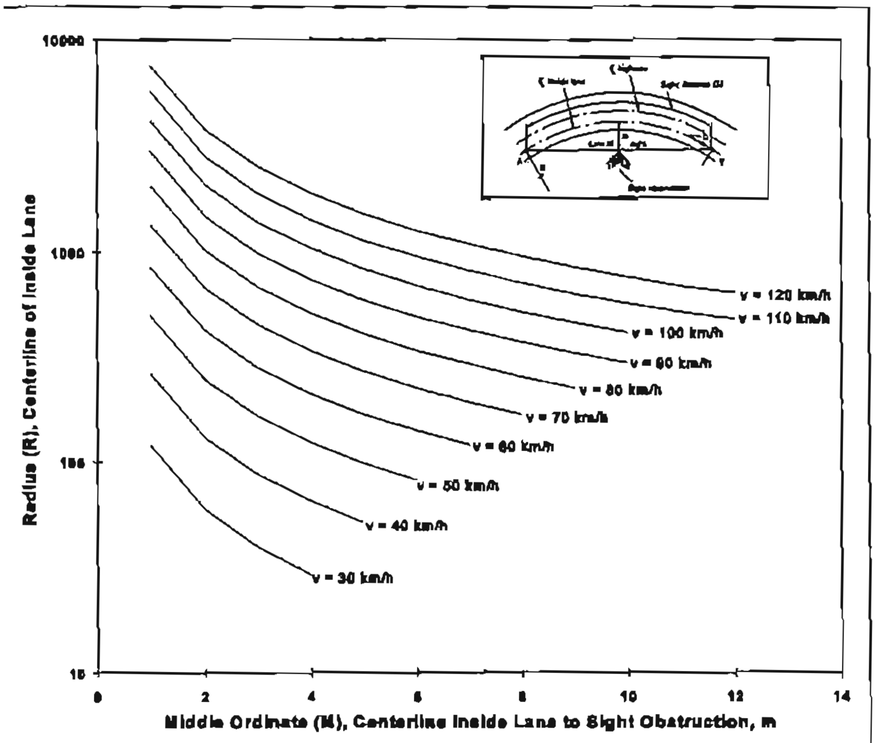
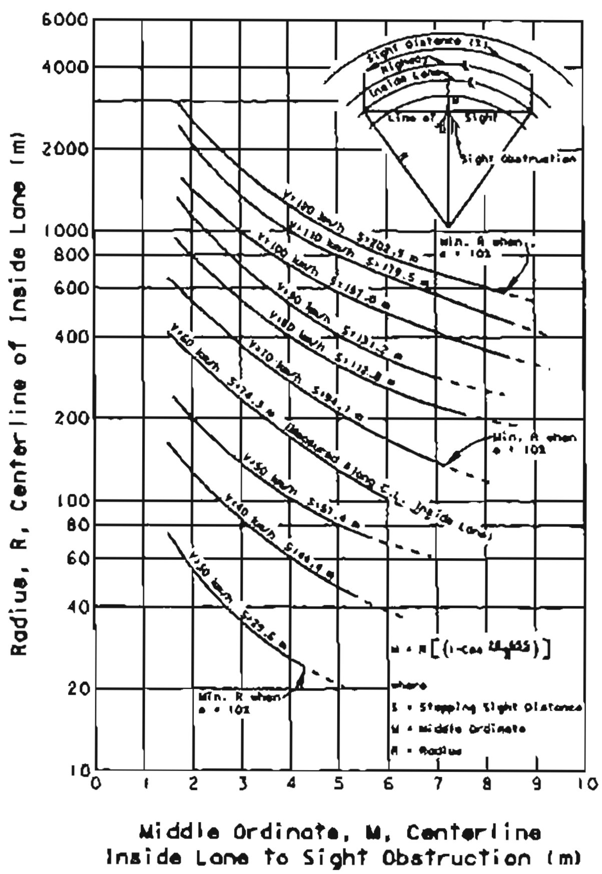


Figure III-24. Relationship Between Radius and Value of Middle Ordinate Necessary to Provide Stopping Sight Distance on Horizontal Curves.

~~established in Figure III-24A, but not the value established in Figure III-24B, will provide acceptable stopping sight distance~~. As was the case with the stopping sight distances in Table III-1, ~~a value at or approaching the upper limit~~ **these values** should be used as a minimum wherever conditions permit because of the increased safety that is provided.

The values ~~at or approaching the upper limit is~~ **in Figure III-24 are** an application of geometry for the several dimensions, as indicated in the diagrammatic sketch and formulas on the figures. These formulas apply only to circular curves longer than the sight distance for the pertinent design speed. For any design speed the relation of R to M is a straight line. Relations of R, M, and V in these



~~Figure III-24(B).~~ **~~Range of lower values—relation between radius and value of middle ordinate necessary to provide stopping sight distance on horizontal curves.~~**

chart forms can be quickly checked. For example, with an 80-km/h design speed and a curve with a ~~350~~ **300** m radius, a clear sight distance with a middle ordinate ~~between 5.3 m (lower value) and 7.5 m (upper value)~~ **6.8** m is needed for stopping sight distance. ~~As another example, for a sight obstruction condition with M = 6.0 m on a curve with a 175 m radius, the resulting sight distance is approximately at the upper value of the range for a speed of 60 km/h.~~

NOTE: NUMBERS IN FOLLOWING EXAMPLE MAY CHANGE.

Horizontal sight restrictions may occur where there is a cut slope on the inside of the curve. For the height criteria used for stopping sight distance of ~~1070~~ **1080**-mm height of eye and ~~150~~ **800**-mm height of object, a height of ~~600~~ **840**-mm may be used as the midpoint of the sight line where the cut slope usually obstructs sight. This assumes that there is little or no vertical curvature. For a highway with a 6.6-m traveled way, 1.8 m shoulders, 0.6-m ditch section, and 1:2 cut slopes, the sight obstruction is about 5.5 m outside the centerline of the inner lane. This is sufficient for adequate sight distance at 50 km/h when curves have a radius of about 80 m or more and at 80 km/h when curves have a radius of about 300 m or more. Curves sharper than these would require flatter slopes, benching, or other adjustments. At the other extreme, highways with normal lateral dimensions of more than 9 m provide adequate stopping sight distances at curves over the entire range of design speeds and curves.

In some instances retaining walls, concrete median safety barriers, and other similar features constructed on the inside of curves may be obstructions and must be checked for stopping sight distance adequacy. As an example, an obstruction of this type offset 1.2 m from the inside edge-of-traveled way has a middle ordinate of about 3.0 m. At 80 km/h this provides adequate sight distance when curves have radius of about 550 m or more. If the offset width is increased to 3.3 m, a curve with a 320 m (or more) radius provides adequate sight distance at the same 80-km/h speed. The same would be true for existing buildings or similar obstructions on the inside of curves.

When the needed stopping sight distance would not be available because the railing or a longitudinal barrier constitutes the obstruction, alternatives should be considered for both safety and economic reasons. The alternates are: increase the offset to the obstruction, increase the radius or reduce the design speed. However, any alternative selected should not require the width of the shoulder on the inside of the curve to exceed 3.6 m because the potential exists that drivers will use the shoulders in excess of that width as a passing or travel lane.

As can be seen from Figure III~~-24A and III-24B,~~ the method presented is only exact when both the vehicle and the sight obstruction are located within the limits of the simple horizontal curve. When either the vehicle or the sight obstruction is situated beyond the limits of the simple curve, the values obtained are only approximate. The same is true if either the vehicle or the sight obstruction, or both, is situated within the limits of spiral or a compound curve. In these instances, the value obtained would result in middle ordinate values slightly larger than those needed to satisfy the selected stopping sight distance. In many instances, the resulting additional clearance will not be significant. Whenever Figures III-24~~A and III-24B~~ are not applicable, it is advisable to check the designs either by utilizing graphical procedures or by utilizing a computational method. Reference (59) provides a method for computing the needed values.

Vertical Curves

General Considerations

Vertical curves to effect gradual change between tangent grades may be any one of the crest or sag types depicted in Figure III-38. Vertical curves should be simple in application and should result in a design that is safe, comfortable in operation, pleasing in appearance, and adequate for drainage. The major control for safe operation on crest vertical curves is the provision of ample sight distances for the design speed. Minimum stopping sight distance should be provided in all cases. Wherever economically and physically feasible, more liberal stopping sight distances should be used. Further additional sight distance should be provided at decision points.

Consideration of motorists' comfort requires that the rate of change of grade be kept within tolerable limits. This consideration is most important in sage vertical curves where gravitational and vertical centrifugal forces act in the same direction. Appearance also should be considered. A long curve has a more pleasing appearance than a short one, which may give the appearance of a sudden break in the profile due to the effect of foreshortening.

Drainage of curbed roadways on sag vertical cures, Type III, Figure III-38, requires careful profile design to retain a gradeline of not less than 0.5 percent or, in some cases, 0.30 percent for the outer edges of the roadway. Although not desirable, flatter grades may be necessary in extenuating circumstances.

For simplicity the parabolic curve with an equivalent vertical axis centered on the vertical point of intersection (VPI) is usually used in roadway profile design. The vertical offsets from the tangent vary as the square of the horizontal distance from the curve end (point of tangency). The vertical offset from the tangent grade at any point along the curve is calculated as a proportion of the vertical offset at the VPI, which is AL/800, where the symbols are shown in Figure III-38. The rate of change of a grade at successive points on the curve is a constant amount for equal increments of horizontal distance, and equals the algebraic difference between intersection tangent grades divided by the length of curve in meters, of A/L in percent per meter. The reciprocal L/A is the horizontal distance in meters required to effect a 1 percent change in gradient and is, therefore, a measure of curvature. The quantity L/A, termed "K," is useful in determining the horizontal distance from the vertical point of curvature (VPC) to the apex of Type I curve or to the low point of Type III curves. This point where the slope is zero occurs at a distance from the VPC equal to K times the approach gradient. The K value is also useful in determining minimum lengths of vertical curves for various design speeds. Other details on parabolic vertical curves are found in textbooks on highway engineering.

On certain occasions, because of critical clearance or other controls, the use of unsymmetrical vertical curves may be required. Because the conditions dictating the need for these curves are infrequent, the derivation and use of the appropriate formulas have not been included herein. For use in such limited instances, refer to unsymmetrical curve data found in a number of highway engineering texts.

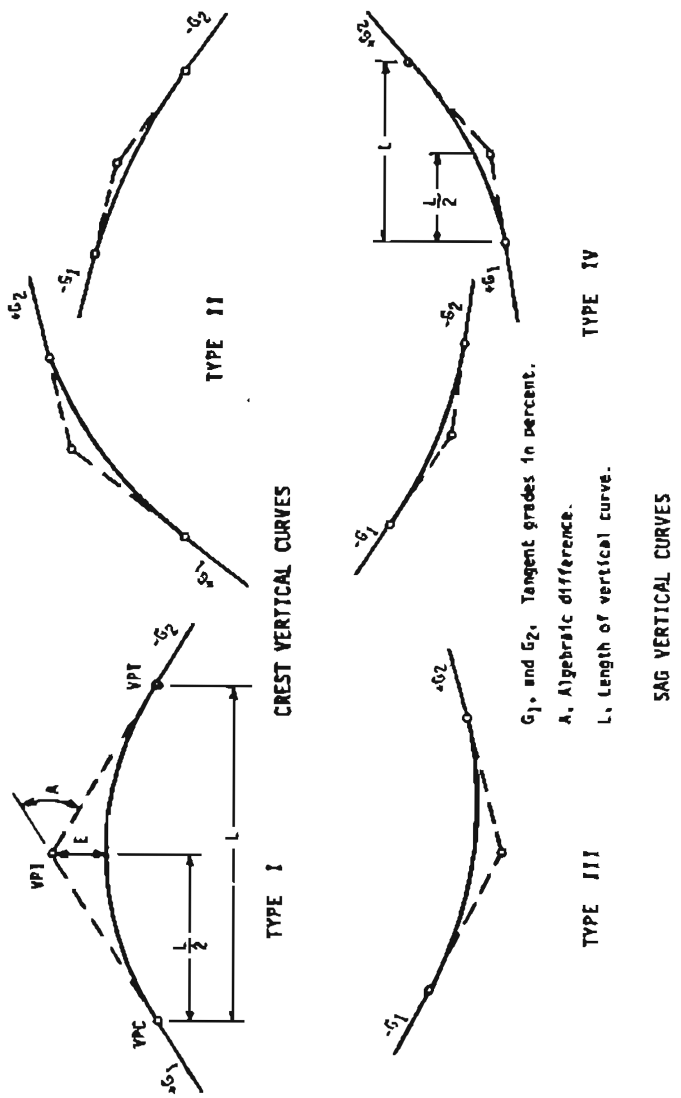


Figure III-38. Types of vertical curves.

Crest Vertical Curves

Minimum lengths of crest vertical curves as determined by sight distance requirements generally are satisfactory from the standpoint of safety, comfort, and appearance. An exception may be at decision areas, such as sight distance to ramp exit gores, where longer lengths are necessary. Refer to the section in this chapter concerning decision sight distance.

The basic formulas for length of a parabolic vertical curve in terms of algebraic difference in grade and sight distance follow:

When S is less than L,

$$L = \frac{AS^2}{100(\sqrt{2h_1} + \sqrt{2h_2})^2} \qquad (1)$$

When S is greater than L,

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

where: L = length of vertical curve, m;
S = sight distance, m;
A = algebraic difference in grades, percent;
h_1 = height of eye above roadway surface, m (normally ~~1070~~ **1080** mm/1000 mm/m)
h_2 = height object above roadway surface, m (normally ~~150~~ **600** mm/1000 mm/m)

When the height of eye and the height of object are ~~1070~~ **1080mm** and ~~150~~ **600** mm, respectively, as used for stopping sight distance.

When S is less than L,

$$L = \frac{AS^2}{\cancel{404}\ 658} \qquad (3)$$

When S is greater than L,

$$L = 2S - \frac{\cancel{404}\ 658}{A} \qquad (4)$$

Design controls - stopping sight distance. The required lengths of vertical curves from formulas 3 and 4 for different values of A to provide the ~~upper value of the range~~ of stopping sight distances for each design speed are shown in Figure III-39. The solid lines give the required lengths, on the basis of rounded values of K as determined from these equations. ~~The dotted line for K = 39.39 gives unrounded values for 70 km/h for the comparison.~~

The short dashed curve at the lower left, crossing these lines, indicates where S= L. Note that to the right of the S=L line, the value of K, or length of vertical curve per percent change in A, is a simple and convenient expression of the design control. For each design speed this single value is a positive whole number that is indicative of the rate of vertical curvature. The design control in terms of K covers all combinations of A and L for any one design speed; thus, A and L need not be indicated separately in a design value tabulation. The selection of design curves is facilitated because the required length of curve in meters is equal to K times the algebraic difference in grades in percent L=KA. Conversely, the checking of plans is simplified by comparing all curves with the design K value.

Table III-35 shows the computed K values for lengths of vertical curves as required for the ~~range of~~ values of stopping sight distances, Table III-1, for each design speed. For direct use in design, values of K are rounded as shown in the right column. The ~~upper~~, rounded values of K are plotted as the solid lines in Figure III-39. Rounded values of K are higher than computed values, but the differences are not significant.

Where S is greater than L (lower left in Figure III-39), computed values plot as a curve (as shown by the dashed line for 70 km/h) that bends to the left, and for small values of A the required lengths are zero because the sight line passes over the apex. This relation does not represent desirable design practice. Most of the States use a minimum length of vertical curve, expressed as either a single value, a range for different design speeds, or a function of A. Values now in use range form about 30 to 100 m. To recognize the distinction in design speed and to approximate the range of current practice, minimum lengths of vertical curves are expressed as about 0.6 times the design speed, or L_{min} = 0.6V where V is in kilometers per hour and L is in meters. These terminal adjustments show as the vertical lines at the lower left of Figure III-39.

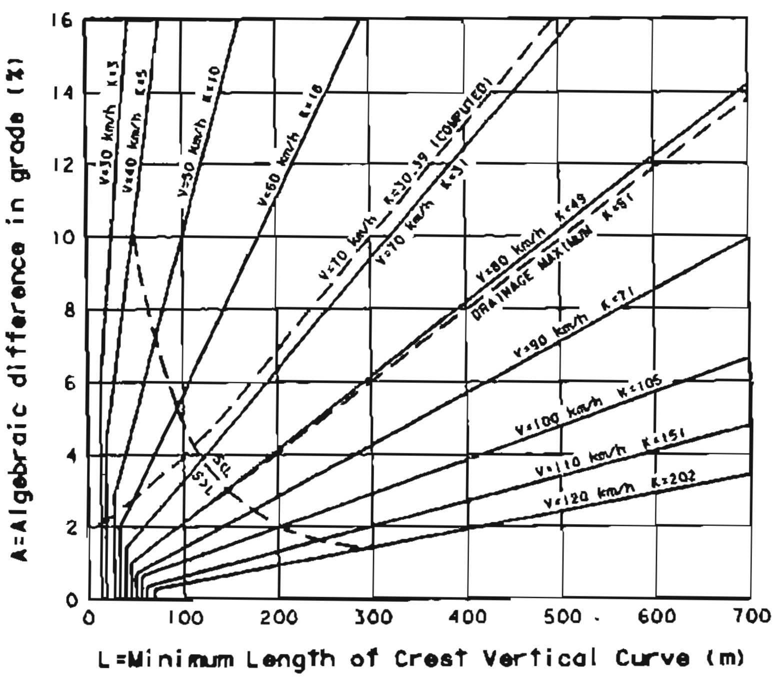
~~Design Speed (km/h)~~	~~Assumed Speed for Condition (km/h)~~	~~Coefficient of Friction f~~	~~Stopping Sight Distance for Design (m)~~	~~Rate of Vertical Curvature, K (length (m) per % of A)~~	
				~~Computed~~	~~Rounded for Design~~
~~30~~	~~30-30~~	~~0.40~~	~~29.6-29.6~~	~~2.17-2.17~~	~~3-3~~
~~40~~	~~40-40~~	~~0.38~~	~~44.4-44.4~~	~~4.88-4.88~~	~~5-5~~
~~50~~	~~47-50~~	~~0.35~~	~~57.4-62.8~~	~~8.16-9.76~~	~~9-10~~
~~60~~	~~55-60~~	~~0.33~~	~~74.3-84.6~~	~~13.66-17.72~~	~~14-18~~
~~70~~	~~63-70~~	~~0.31~~	~~94.1-110.8~~	~~21.92-30.39~~	~~22-31~~
~~80~~	~~70-80~~	~~0.30~~	~~112.8-139.4~~	~~31.49-48.10~~	~~32-49~~
~~90~~	~~77-90~~	~~0.30~~	~~131.2-168.7~~	~~42.61-70.44~~	~~43-71~~
~~100~~	~~85-100~~	~~0.29~~	~~157.0-205.0~~	~~61.01-104.02~~	~~62-105~~
~~110~~	~~91-110~~	~~0.28~~	~~179.5-246.4~~	~~79.75-150.28~~	~~80-151~~
~~120~~	~~98-120~~	~~0.28~~	~~202.9-285.6~~	~~101.90-201.90~~	~~102-202~~

~~**Table III-35. Design controls for crest vertical curves.**~~

Initial Speed (km/h)	Stopping Sight Distance for Design (m)	Rate of Vertical Curvature, K [length (m) per % of A]	
		Crest Curves	Sag Curves
30	31.0	2	4
40	45.9	4	8
50	63.1	7	12
60	82.5	11	17
70	104.2	17	23
80	128.2	25	29
90	154.4	37	37
100	182.9	51	45
110	213.7	70	53
120	246.7	93	62

TABLE III-35. Recommended Design Controls for Vertical Curves

Continued -



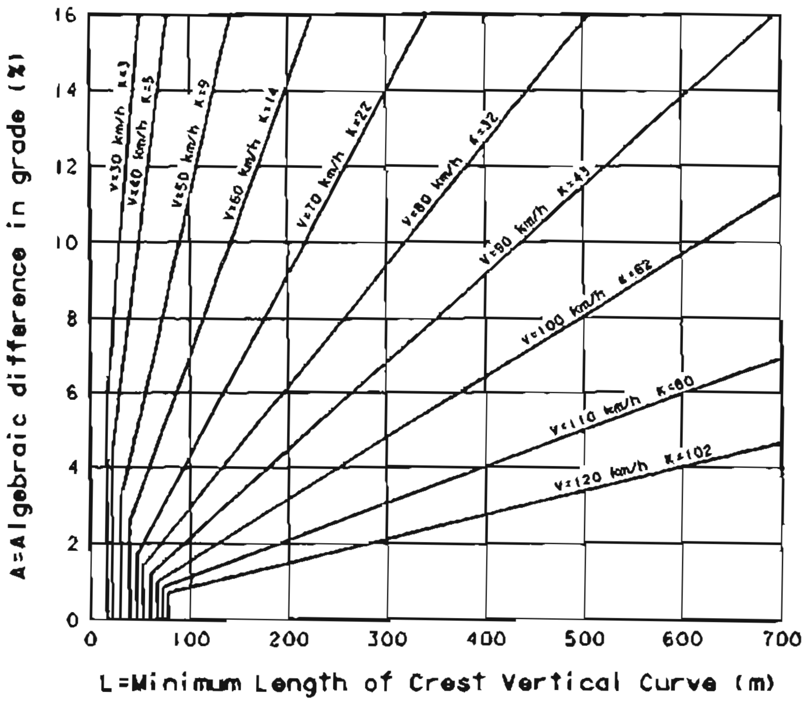
~~**Figure III-39. Design controls for crest vertical curves, for stopping sight distance - upper range.**~~

NOTE: FIGURE TO BE REDONE.

The above values of K derived when S is less than L also can be used without significant error where S is greater than L. As shown in Figure III-39, extension of the diagonal lines to meet the vertical lines for minimum lengths of vertical curves results in appreciable differences from the theoretical only where A is small and little or no additional cost is involved in obtaining longer vertical curves.

~~The lower K factors for each speed developed in Table III-35 were based on the assumption that most vehicles reduce their speed in inclement weather or on wet pavements. As discussed in greater detail in the section "Sight Distance" in this chapter, studies conducted in recent years of traffic on wet and dry pavements fail to support this stated hypothesis. Nevertheless, a range of values of stopping sight distance have been computed based on friction factors for wet pavements and on vehicular speeds equal to the average running speed and the design speed of the highway. Figure III-40 provides the required lengths of vertical curves for different values of A to provide the lower range of stopping sight distances for each design speed (based on the corresponding running speed).~~

For night driving on highways without lighting, the length of visible roadway is that roadway which is directly illuminated by the headlights of the vehicle. For certain conditions, the minimum stopping sight distance values used for design exceed the length of visible roadway. First, vehicle headlights have limitations on the projection distance for the light intensity levels that are required for visibility purposes. When headlights are operated on low-beam, the reduced candlepower at the source plus the downward projection angle significantly restrict the length of visible roadway surface. Thus, particularly for high-speed conditions, stopping sight distance values exceed road-surface visibility distances afforded by the low-beam headlights regardless of whether the roadway profile is level or vertically curving. Secondly, for crest vertical curves the area forward of the headlight beam point of tangency with the roadway surface is shadowed and receives only indirect illumination. Since the headlight mounting height (typically about 600 mm) is lower than the driver eye height (~~1070~~ **1080** mm for design), the sight distance to an illuminated object ~~150~~ **600** mm in height is controlled by the height of the vehicle headlights rather than by the direct line of sight. Any object within the shadow zone must be high enough to extend into the headlight beam to be directly illuminated. On the basis of formula 1, the bottom of the headlight beam is about 400 mm above the roadway at a distance ahead of the vehicle equal to the low value of the range of stopping sight distance. Although the vehicle headlight system does limit roadway visibility length as mentioned above, there is some mitigating effect in that other vehicles, whose taillight height typically varies from 450 to 600 mm, and other sizable hazardous objects receive direct lighting from headlights at stopping sight distance values used for design. It also may be rationalized that drivers are aware that visibility at night is less than during the day regardless of road and street design features, and that vehicle operators are thus more attentive and alert.



~~Figure III-40. Design controls for crest vertical curves for stopping sight distance—lower range.~~

There is a level point of minute length on a crest vertical curve of Type I (Figure III-38), but no difficulty with drainage on highways with curbs is experienced if the curve is sharp enough so that a minimum grade of 0.30 percent is reached at a point about 15 m from the crest. This corresponds to a 51-m per percent change in grade; this line is plotted in Figure III-39 as the drainage maximum. All combinations above or to the left of this line would satisfy this criterion for drainage. The combinations below and to the right of this line involve flatter vertical curves. Special attention is needed in these cases to ensure proper pavement drainage near the apex of crest vertical curves. It is not intended that a K value of 51 be considered a design maximum, but merely the value beyond which drainage must be more carefully designed.

Sag Vertical Curves

At least four different criteria for establishing lengths of sag vertical are recognized to some extent. These are (1) headlight sight distance, (2) rider comfort, (3) drainage control, and (4) a rule-of-thumb for general appearance.

Headlight sight distance has been used directly by some authorities and for the most part is the basis for determining the length of sight distance used herein. When a vehicle traverses a sag vertical curve at night, the proportion of highway lighted ahead is dependent on the position of the headlights and the direction of the light beam. General use is being given to a headlight height of 600 mm and a 1° upward divergence of the light beam from the longitudinal axis of the vehicle. The upward spread of the light beam provides some additional visible length but this is generally ignored. The following formulas show the S, L, and A relation, using S as the distance between the vehicle and point where the 1° angle of light ray intersects the surface of the roadway:

When S is less than L,

$$L = \frac{AS^2}{200[0.6 + S(\tan 1°)]} = \frac{AS^2}{120 + 3.5S} \qquad (7)$$

When S is greater than L,

$$L = 2S - \frac{200[0.6 + S(\tan 1°)]}{A} = 2S - \left(\frac{120 + 3.5S}{A}\right) \qquad (8)$$

where: L = length of sag vertical curve, m;
S = light beam distance, m; and
A = algebraic difference in grades, percent.

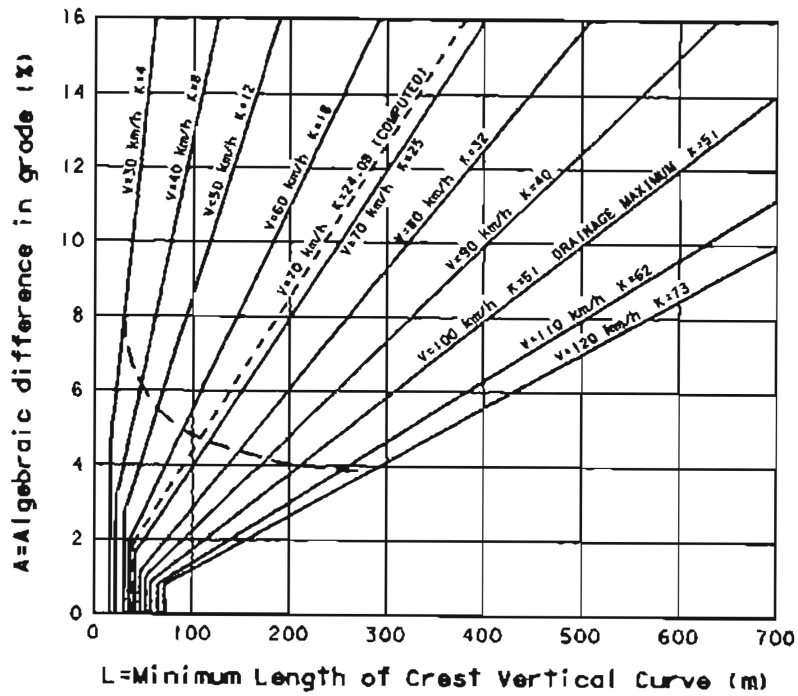
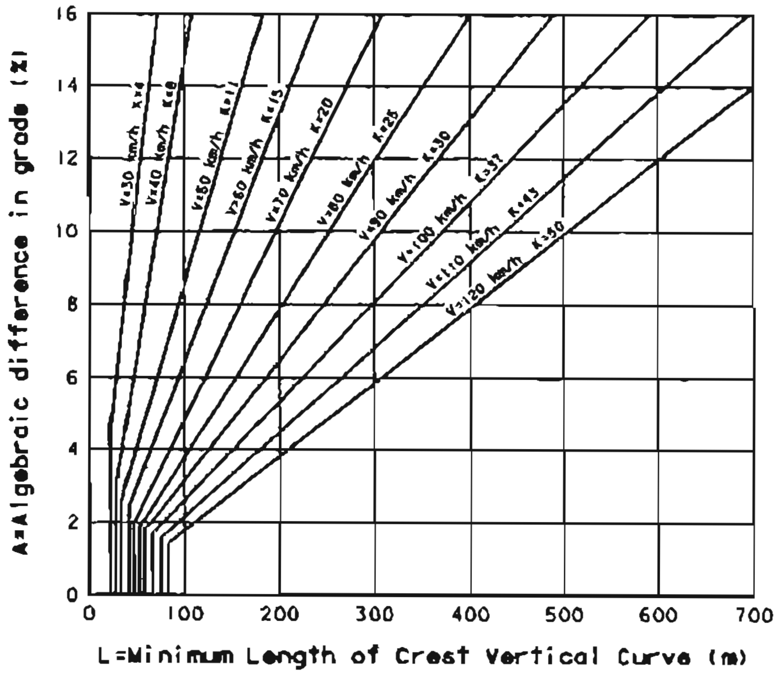


Figure III-41. Design controls for sag vertical curves~~—upper range~~.

NOTE: FIGURE TO BE REDONE.



~~Figure III-42. Design controls for sag vertical curves—lower range.~~

For overall safety on highways, a sag vertical curve should be long enough so that the light beam distance is nearly the same as the stopping sight distance. Accordingly, it is pertinent to use stopping sight distances for different design speeds as the S value in the above formulas. The resulting lengths of vertical curves for the ~~upper value of the range of~~ stopping sight distances for each design speed are shown in Figure III-41 with solid lines using rounded K values as was done for crest vertical curves. ~~and the dotted line for K = 26.6 being an unrounded value for 70 km/h for comparison. Figure III-42 provides the lengths of sag vertical curves for various algebraic differences in grades for the lower range of stopping sight distance.~~

The comfort effect of change in vertical direction is greater on sag than on crest vertical curves because gravitational and centrifugal forces are combining rather than opposing forces. Comfort due to change in vertical direction is not measured readily because it is affected appreciably by vehicle body suspension, tire flexibility, mass carried, and other factors. The limited attempts at such measurements have led to the broad conclusion that riding is comfortable on sag vertical curves when the centrifugal acceleration does not exceed 0.3 m/s^2. The general expression for such a criterion is:

$$L = \frac{AV^2}{395}$$

where L and A are the same as in previous formulas, and V is the design speed, km/h.

The length of vertical curve required to satisfy this comfort factor at the various design speeds is only about 50 percent of that required to satisfy the headlight sight distance requirement for the normal range of design conditions.

Drainage affects design of vertical curves of Type III (Figure III-38) where curbed sections are used. An approximate criterion for sag vertical curves is the same as that expressed for the crest conditions, that is, providing a minimum grade of 0.30 percent within 15 m of the level point. This criterion plots the same or very close to the same as the line shown in Figure III-41 for the 100 km/h, K = 51. The drainage requirement differs from other criteria in that the length for any other criterion is a minimum. The maximum length of the drainage criterion is greater than the minimum length for other criteria up to 100 km/h and is nearly equal for other criteria up to 120 km/h for minimum-length vertical curves.

For general appearance, some use formerly was made of rule-of-thumb for length of sag vertical curves wherein the minimum value of L is 30A or, in Figure III-41, K = 30. This approximation is a generalized control for small or intermediate values of A. Compared with headlight sight distance, it corresponds to a design speed between 70 and 80 km/h. On high-type highways longer curves are deemed appropriate to improve appearance.

From the preceding it is evident that design controls for sag vertical curves differ from those for crests, and separate design values are needed. The headlight sight distance basis appears to be the most logical for general use, and the values determined for stopping sight distances are within the limits recognized in current practice. It is concluded to use this criterion to establish design values for a range of lengths of sag vertical curves. As in the case of crest vertical curves, it is convenient to express the design control in terms of the K rate for all values of A. This entails some deviation from the computed values for small values of A, but the differences are not significant. Table III-35 shows the ~~range of~~ computed values and the rounded values of K selected as design controls. The lengths of sag vertical curves on the basis of the design speed values of K are shown by the solid lines in Figure III-41. It is to be emphasized that these lengths are ~~minimum values~~ based on design speed; longer curves are desired wherever feasible, but special attention to drainage must be exercised where a K value in excess of 51 is used.

~~Assumed Design Speed (km/h)~~	~~Speed for Condition (km/h)~~	~~Coefficient of Friction f~~	~~Stopping Sight Distance for Design (m)~~	~~Rate of Vertical Curvature, K (length (m) per % of A~~	
				~~Computed~~	~~Rounded for Design~~
~~30~~	~~30-30~~	~~0.40~~	~~29.6-29.6~~	~~3.88-3.88~~	~~4-4~~
~~40~~	~~40-40~~	~~0.38~~	~~44.4-44.4~~	~~7.11-7.11~~	~~8-8~~
~~50~~	~~47-50~~	~~0.35~~	~~57.4-62.8~~	~~10.20-11.54~~	~~11-12~~
~~60~~	~~55-60~~	~~0.33~~	~~74.3-84.6~~	~~14.45-17.12~~	~~15-18~~
~~70~~	~~63-70~~	~~0.31~~	~~94.1-110.8~~	~~19.62-24.08~~	~~20-25~~
~~80~~	~~70-80~~	~~0.30~~	~~112.8-139.4~~	~~24.62-31.86~~	~~25-32~~
~~90~~	~~77-90~~	~~0.30~~	~~131.2-168.7~~	~~29.62-39.95~~	~~30-40~~
~~100~~	~~85-100~~	~~0.29~~	~~157.0-205.0~~	~~36.71-50.06~~	~~37-51~~
~~110~~	~~91-110~~	~~0.28~~	~~179.5-246.4~~	~~42.95-61.68~~	~~43-62~~
~~120~~	~~98-120~~	~~0.28~~	~~202.9-285.6~~	~~49.47-72.72~~	~~50-73~~

~~Table III-37. Design controls for sag vertical curves.~~

Minimum lengths of vertical curves for flat gradients also are recognized for sag conditions. The values determined for crest conditions appear to be generally suitable for sags. Lengths of sag vertical curves, shown as vertical lines in Figure III-41, are equal to 0.6 times the design speed.

Sag vertical curves shorter than the length computed from Table III-35 may be justified for economic reasons in cases where an existing element, such as a structure not ready for replacement, controls the vertical profile. In certain cases ramps may also be designed with shorter sag vertical curves. Fixed source lighting is desirable in these cases. For street design, some engineers accept design of a sag or crest where A is about 1 percent or less without a length of calculated vertical curve. However, field modifications during construction usually result in constructing the equivalent to a vertical curve, even if short.

TABELE I-1. NECESSARY REVISIONS TO THE AASHTO GREEN BOOK

Page	Description
28	Paragraph 4 to be revised to include the use of deceleration rates used to calculate stopping sight distances
62	Section entitled "Operating Speed" to be revised as indicated in Appendix I (pg. I-2).
62	First paragraph of section entitled "Design Speed" to be revised as indicated in Appendix I (pg. I-2).
63	Third paragraph to be revised as indicated in Appendix I (pg. I-3).
63	Fifth paragraph to be revised as indicated in Appendix I (pg. I-3).
65	Fifth paragraph to be revised as indicated in Appendix I (pg. I-5).
66	Second paragraph to be revised as indicated in Appendix I (pg. I-5).
67	Second paragraph to be revised as indicated in Appendix I (pg. I-6).
67	Fourth paragraph to be revised as indicated in Appendix I (pg. I-6).
68	Fifth paragraph to be revised as indicated in Appendix I (pg. I-7).
69	Second paragraph to be revised as indicated in Appendix I (pg. I-8).
119	Second paragraph to be revised as indicated in Appendix I (pg. I-11).
119	Formula to be revised as indicated in Appendix I (pg. I-12).
119-123	Text from final paragraph on page 120 to second paragraph on page 123 to be deleted and replaced with text as indicated in Appendix I (pg. I-13).
120	Table III-1 to be replaced with Table 58 as indicated in Appendix I (pg. I-15).
122	Figure III-1 to be deleted. Revise remaining figure numbers after deletion of this figure.
123	Formula to be revised as indicated in Appendix I (pg. I-17).
123	Final paragraph to be revised as indicated in Appendix I (pg. I-17).
124	Second paragraph to be revised as indicated in Appendix I (pg. I-17).
124	Formula to be revised as indicated in Appendix I (pg. I-17).
124	Third paragraph to be revised as indicated in Appendix I (pg. I-18).

Page	Description
125	Table III-2 to be revised.
125	First paragraph to be revised as indicated in Appendix I (pg. I-18).
127	Table III-3 values to be revised based on revised object and eye heights.
127	Third paragraph to be revised to represent revised object and eye heights.
136	Final paragraph to be revised as indicated in Appendix I (pg. I-19).
137	Paragraph entitled " Stopping sight distance object" to be revised as indicated in Appendix I (pg. I-19).
138	Final paragraph to be revised as indicated in Appendix I (pg. I-21).
139	Second paragraph to be revised as indicated in Appendix I (pg. I-21).
140	Figure III-3 to be revised to indicate the revised object and eye heights for both stopping and passing sight distances.
219-223	Section entitled "Stopping sight distance" to be revised as indicated in Appendix I (pg. I-23 to I-26).
220	Figure III-24 to be revised.
221	Figure III-25 to be deleted.
223	Section entitled "Passing sight distance" to be revised to indicate the revised driver eye height.
282	Revise values of h_1 and h_2 in equations (1) and (2). In the paragraph that follows revise the object and driver eye heights as indicated in Appendix I (pg. I-29).
282-283	Revise formulae (3) and (4) as indicated in Appendix I (pg. I-29).
283-288	Section entitled "Design Controls-stopping sight distance" to be revised as indicated in Appendix I (pg. I-30 to I-33)
284	Table III-35 to be revised as indicated in Appendix I (pg. I-31).
284	Figure III-39 to be revised.
287	Formulae (5) and (6) must be revised to incorporate new driver eye height.
287	Table III-36 to be revised to incorporate new formulae for equations (5) and (6).
288-293	Section entitled "Sag vertical curves" to be revised as indicated in Appendix I (pg. I-34 to I-37).

Page	Description
289	Figure III-41 to be revised.
290	Figure III-42 to be deleted.
292	Table III-37 to be replaced by Table 60 from "Final Draft Report".
418	Section entitled "Sight Distance"to be revised to incorporate revised object and eye heights.
419	Table V-2 to be revised to reflect revised object and eye heights.
420	Table V-3 to be revised to reflect revised eye height.
427	Table V-9 to be revised to reflect revised eye height
429	Revise stopping sight distance range referred to in paragraph entitled "Sight Distance".
446	Section entitled "Sight Distance" to be revised to incorporate revised object and eye heights.
446	Table V-11 to be revised to incorporate revised object and eye heights.
447	Table V-12 to be revised to incorporate revised driver eye height.
461	Section entitled "Sight Distance" to be revised to incorporate revised object and eye heights.
462	Table VI-2A to be revised to incorporate revised object and eye heights.
462	Table VI-2B to be revised to incorporate revised driver eye height.
472	Review stopping sight distance range referred to in paragraph entitled "Sight Distance"
490	Table VII-3 to be revised to incorporate revised eye and objects heights. Check legend of last column (PSD?)
710	Second paragraph to be revised to incorporate revised driver eye height.
710	Figure IX-38 to be revised.
715	Figure IX-41 to be revised.
722	Section entitled "General Considerations" to be revised to replace coefficients of friction formulation with revised deceleration rate formulation.

Page	Description
722	Table IX-10 to be revised. (SSD values)
722	Table IX-11 to be revised.(SSD values)
723	Section entitled "General Considerations" to be revised to incorporate new driver eye and object heights.
724	Figure IX-44 to be revised. (SSD values)
725	Figure IX-45 to be revised. (SSD values)
797-801	Formulae, discussion, Table IX-21 and Figures IX-78,79 regarding sight distance calculation to be reviewed due to the change in the formulation of the calculation of Stopping sight distance.