A Safety Evaluation of Current Design Criteria for Stopping Sight Distance

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This paper presents a review of the current AASHO design standards and an evaluation of these standards based on existing practices. The evaluation considered the criteria that were employed in developing the standards and that include driver perception-reaction time, design friction factors, assumed speeds for design, driver's eye height, and object height. In addition, the report proposes a new philosophy for sight distance design, a philosophy that considers the visual requirements for safety depending on operational conditions.

•ABILITY TO SEE THE ROADWAY ahead is of the utmost importance in the safe and efficient operation of a highway. The path and speed of vehicles on the highway are subject to the control of drivers whose training is largely elementary. If safety is to be built into highways, the design must provide sight distance of sufficient length to permit drivers enough time and distance to control the path and speed of their vehicle, in order to avoid unforeseen collision circumstances.

There has been an increasing concern by highway and traffic engineers regarding the validity of the basic criteria that are fundamental to geometric design standards. The design standards for stopping sight distance employed by most state highway departments are taken from "A Policy on Geometric Design of Rural Highways" published by the American Association of State Highway Officials (1). The design criteria for stopping sight distance presented by AASHO are based on studies conducted between 1934 and 1953. As such, they may no longer be representative because vehicle, roadway, and driver characteristics have changed. In addition, there are uncertainties regarding the assumptions employed in establishing the safe stopping distance design standards.

This research study was addressed to an evaluation of the validity of the AASHO standards for safe stopping sight distance. The method of study employed a comprehensive review of current stopping sight distance standards and an evaluation of their validity, based on an analysis of existing practices.

STOPPING SIGHT DISTANCE DESIGN STANDARDS

A comprehensive description of the AASHO highway design standards on stopping sight distance is offered as a basis for an evaluation of their validity (1, pp. 134-140, 147-149).

Stopping Sight Distance Defined

The AASHO Policy defines sight distance as "the length of highway ahead visible to the driver." The minimum sight distances available should be sufficiently long to enable a vehicle traveling at or near the likely top speed to stop before reaching an object in its path. Although greater length is desirable, sight distance at every point along the highway should be at least that required for a below-average driver or vehicle to stop. Minimum stopping sight distance is the sum of 2 distances: (a) the distance traveled by the vehicle during the period of perception and brake reaction and (b) the distance required to brake the vehicle to a stop.

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Brake Reaction Time

Many studies have been conducted to determine the brake reaction time of drivers. These studies show that the brake reaction time for most people is from 0.5 to 0.7 sec (1). Some drivers react in a shorter time and some require a full second or more. One of the primary variables is age of the driver; as the driver becomes older, his reaction time becomes greater (2). The AASHO Policy states: "For safety, a reaction time that is sufficient for most operators, rather than for the average operator, should be used in any determination of minimum sight distance. A brake reaction time of a full second is assumed herein."

Perception Time

Perception time, as considered here, is "the time required for a driver to perceive the need for brake application." It is the time lapse from the instant an object is visible to the driver to the instant he realizes that the object is in his path and that a stop is required. Little is known about the exact time required for driver perception. It varies with the ability of the driver, his emotional and physical condition, and the visibility of the object. At high speeds, perception time may be less than at low speeds because the driver is more alert; however, the longer distances associated with higher speeds may require more time because of the degradation in visual acuity associated with higher speeds (3).

Perception-Reaction Time

Research data on perception time are very limited. Most available data combine perception time with brake reaction time. One study (4) conducted with alerted drivers determined an average combined value of 0.64 sec, with 5 percent of the drivers requiring over 1 sec. Under such conditions, perception time can be expected to be a small portion of the total perception-reaction time. The study concluded that the driver requiring 0.2 to 0.3 sec of perception time would require 1.5 sec for normal highway conditions. In another study (5) with alerted drivers, combined values ranged from 0.4 to 1.7 sec. Supplemental unpublished data in a study (6) of passing maneuvers showed that a perception time of approximately 1 sec was required for drivers to analyze and begin a passing maneuver.

The AASHO Policy considers the data from these studies in arriving at a value for perception-reaction time for use in stopping sight distance design. It states:

> A significant feature of these comparative tests is that the total perception and brake reaction time for highway conditions may be several times that for laboratory conditions, and it is evident that perception time is greater than brake reaction time. In determination of sight distance for design, the perception time value should be larger than the average for all drivers under normal conditions. It should be large enough to include the time taken by nearly all drivers under most highway conditions. For such use herein it is assumed that the perception time value is 15 seconds, and the total of perception and brake reaction time is 2.5 seconds. Available references do not justify distinction over the range in design speed.

Braking Distance

The approximate braking distance of a vehicle on a level roadway may be determined by using the standard formula

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

where

d = braking distance, ft,
V = initial speed, mph, and
f = coefficient of friction between tires and roadway.

In this formula the coefficient of friction, f, is an equivalent constant value representing the entire speed-change interval from V to zero mph. Measurements show that f is not the same for all speeds (7). It decreases as initial speed increases. It varies

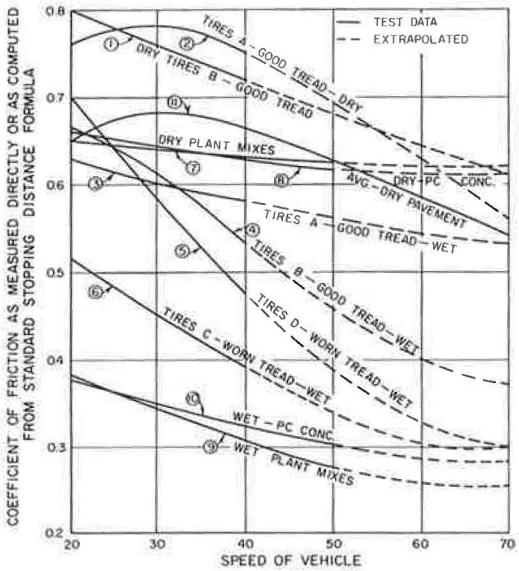


Figure 1. Relationship between friction factor and speed for several conditions.

because of several physical elements such as tire pressure, tire type, tire tread depth, type and condition of pavement, and the presence of water, snow, ice, or mud. These variables are accounted for if the coefficient of friction, f, is computed for each test from the standard formula $d = V^2/30f$. It thus represents the equivalent constant friction factor.

Design Friction Factors

In developing design values for the friction factor, f, AASHO considered the results of several investigators (5, 8, 9). Figure 1 shows the curves relating friction factor, f, to vehicle speed (1, Fig. III-1). For several of these curves, the friction factor, f, was calculated by using the standard formula because 2 of the investigators recorded speed and stopping distance only (5, 8).

Curves 1 to 6 in Figure 1 are from a study (8) in which more than 1,000 measurements of forward stopping distance were made on 32 pavements, both in wet and dry conditions. Several types of tires were used. Curves 7 and 8 are representative of several curves developed in a study (9) that recorded friction factors on 50 surfaces tested when dry by using 3 different methods and 3 types of tires. Curves 9 and 10 from the same study are representative of wet conditions. Curve 11 is the calculated equivalent friction factor for stopping distances, measured (5) on a new high-quality pavement; these were the only tests that included stops from 60 and 70 mph. This curve represents an average of all stops measured.

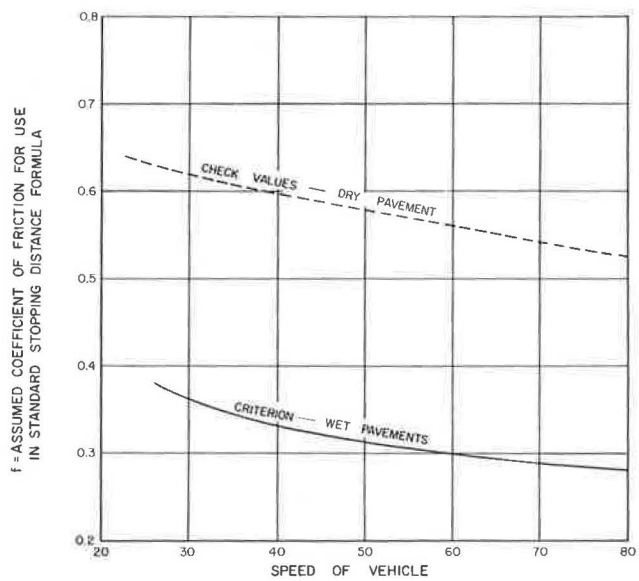


Figure 2. Friction factor values for stopping sight distance design.

The AASHO Policy in concluding its establishment of design values for the friction factor makes the following remarks:

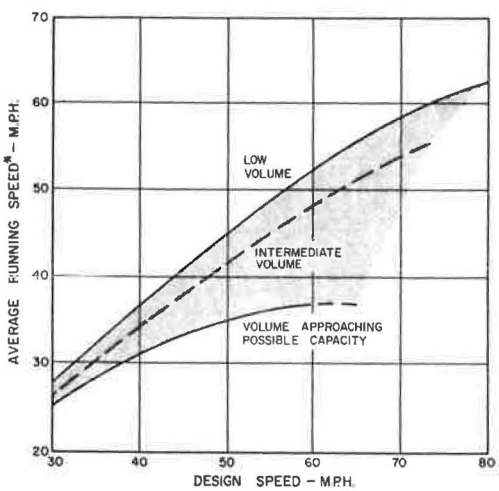
> Because of lower coefficients of friction on wet pavements as compared to dry, the wet condition governs in determining stopping distances for use in design. The coefficients of friction used for design criteria should not only represent wet pavements in good condition but also surfaces throughout their useful life. The values should encompass nearly all significant pavement surface types and the likely field conditions. They should be such as to be safe for worn tires, as well as for new tires, and for nearly all types of treads and tire composition. And, the friction factor should safely encompass the differences in vehicle and driver braking from different speeds. On the other hand, the values need not be so low as to be suitable for obsolescent or bleeding surfaces or for pavements under icy conditions. Preferably, the f values for design should be nearly all inclusive, rather than average; 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 lower curve in Figure III-1B gives the f values assumed for calculation of design stopping distances, recognizing these factors. Comparison with the curves of Figure III-1A shows them to be both practical and conservative.

Figure 2 shows the friction factor values for design referred to in this quotation (1, Fig. III-1B).

Assumed Speed for Conditions

The AASHO Policy states that it is not realistic to assume that travel will occur at full design speed when conditions are wet. It states:

> While the degree to which speeds are lower in inclement weather is not known precisely, it is definite that top speeds will be somewhat lower on wet pavements than on the same pavements in dry weather. For use herein the speed for wet conditions is considered to approximate 80 to 93 percent of design speed which, as previously explained, is indicative of the top speeds when pavements are dry. These speeds are the same as average running speeds for low volume conditions as shown in Figure II-16.



RUNNING SPEED IS THE SPEED (OF AN INDIVIDUAL VEHICLE) OVER A SPECIFIED SECTION OF HIGHWAY, BEING DIVIDED BY RUNNING TIME.

*AVERAGE RUNNING SPEED IS 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.

Figure 3. Relationship between average running speed and design speed.

Figure 3 shows the relationship between average running speed and design speed (1, Fig. II-16). The AASHO Policy refers to data collected to establish these curves but does not give a source, either published or unpublished. The relationship was supposedly established from data that related average spot speed to design speed on horizontal curves.

Minimum Stopping Sight Distance Design Values

The sum of the distance traveled during the perception and brake reaction time and the distance required to stop the vehicle, is the minimum stopping sight distance. Table 1 gives the AASHO Policy design values (and their bases of computation) for minimum stopping sight distance. Comparative check values for dry pavements are shown in the lower part of Table 1. These are computed by use of full design speed and f-values for dry pavements as shown in the upper part of Figure 2.

Effect of Grades on Stopping

When a highway is on a grade, the standard formula for braking distance is

$$d = \frac{V^2}{30\,(f + G)}$$

in which G is the percentage of grade divided by 100, and the other terms are as previously stated. Table 2 gives the extent of the grade corrections of the AASHO design values for stopping sight distance.

Criteria for Measuring Sight Distance

All the material presented previously deals with the design level required for adequate stopping sight distance based on driver and vehicle performance levels. To apply these minimum stopping sight distances in the design procedure requires geometric considerations of the highway alignment (both horizontal and vertical), the height of the

TABLE 1

DESIGN STANDARD FOR MINIMUM STOPPING SIGHT DISTANCE

Design Speed (mph)	Assumed Speed for Condition (mph)	Perception-Reaction		Coefficient of Friction (ft)	Braking Distance on Level (ft)	Stopping Sight Distance	
		Time (sec)	Distance (ft)			Computed (ft)	Rounded for Design (ft)
				Wet Pavements			
30	28	2.5	103	0.36	73	176	200
40	36	2.5	132	0.33	131	263	275
50	44	2.5	161	0.31	208	369	350
60	52	2.5	191	0.30	300	491	475
65	55	2.5	202	0.30	336	538	550
70	58	2.5	213	0.29	387	600	600
75[a]	61	2.5	224	0.28	443	667	675
80[a]	64	2.5	235	0.27	506	741	750
				Comparative Values—Dry Pavements			
30	30	2.5	110	0.62	48	158	
40	40	2.5	147	0.60	89	236	
50	50	2.5	183	0.58	144	327	
60	60	2.5	220	0.56	214	434	
65	65	2.5	238	0.56	251	489	
70	70	2.5	257	0.55	297	554	
75	75	2.5	275	0.54	347	622	
80	80	2.5	293	0.53	403	696	

[a]Design speeds of 75 and 80 mph are applicable only to highways with full control of access or where such control is planned in the future.

TABLE 2

EFFECT OF GRADE ON STOPPING SIGHT DISTANCE FOR WET CONDITIONS

Design Speed (mph)	Assumed Speed for Condition (mph)	Correction in Stopping Distance (ft)					
		Percent Decrease for Upgrades			Percent Increase for Downgrades		
		3	6	9	3	6	9
30	28	—	10	20	10	20	30
40	36	10	20	30	10	30	50
50	44	20	30	—	20	50	—
60	52	30	50	—	30	80	—
65	55	30	60	—	40	90	—
70	58	40	70	—	50	100	—
75[a]	61	50	80	—	60	120	—
80[a]	64	60	90	—	70	150	—

[a]Design speeds of 75 and 80 mph are applicable only to highways with full control of access or where such control is planned in the future.

driver's eye, and the height of the object. Sight distance along a highway is measured from the top of an object on the traveled way when it first comes into view. The general equations used in the design for stopping sight distance over crest vertical curves are

$$L = 2S - \frac{200\,(\sqrt{H_1} + \sqrt{H_2})^2}{A} \quad \text{for } S > L$$

and

$$L = \frac{AS^2}{200\,(\sqrt{H_1} + \sqrt{H_2})^2} \quad \text{for } S < L$$

where

L = length of vertical curve,
S = sight distance, ft,
A = algebraic difference in grade over the crest,
H_1 = height of driver's eye, ft, and
H_2 = height of object, ft.

The equation used in the design for stopping sight distance on horizontal highway curves is

$$S = \frac{R}{28.65} \cos^{-1} \frac{R - m}{R}$$

where

S = sight distance, ft,
R = radius of highway curve, ft, and
m = distance in between obstruction and the centerline of the inside lane.

For more than 20 years, the AASHO design policies (1, 10) based stopping sight distance criteria on a driver's eye height of 4.5 ft above the ground. In 1965, the AASHO Policy adopted a height of 3.75 ft. A study by Stonex (11) probably influenced this decision. Percentile distributions of "average" driver eye heights are shown for automobile models of various years in Figure 4. Median driver eye height has decreased from 56.5 in. in 1936 to 47.5 in. in 1960. Stonex surmised that average driver eye heights would not fall below about 42 in. because of the need for automobiles to conform with sight distance constraints of existing highways.

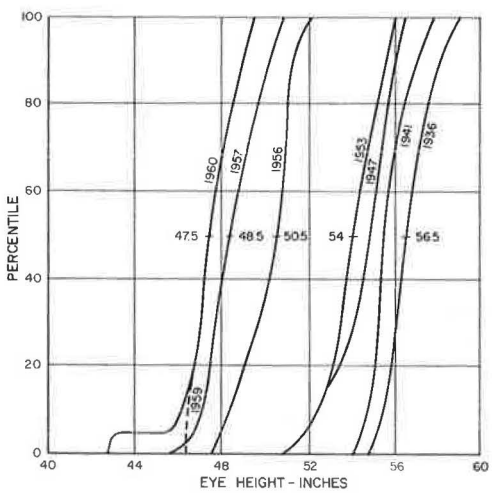


Figure 4. Percentile distribution of driver's eye height for automobile models of various years.

Minimum stopping sight distance is based on the distance required to stop safely from the instant a stationary object in the same lane becomes visible. On crest vertical curves, this point is limited by some point on the road surface. For horizontal curves, it is limited by a lateral obstruction beyond the roadway on the inside of the curve. The height of object that should be used to measure stopping sight distance on crest vertical curves has been a controversial subject (1,10).

The safest height of object would be zero (i.e., the surface of the roadway would be visible to the driver for the full length of the minimum stopping sight distance). Using this criterion, however, could result in long vertical curves requiring considerable excavation cost. The height should not be more than the approximate 2-ft height of vehicle taillights. On the other hand, this height would allow questionably short vertical curves, because lower objects on the road, such as small animals, merchandise dropped from a truck, or rocks rolled from a side cut, may have to be seen to avoid a collision.

Examination of the required lengths of vertical curves for the minimum stopping sight distance (AASHO design values) in conjunction with various heights of objects indicates a significant relationship. The AASHO Policy states:

> Plottings (not shown) of lengths of vertical curves with respect to height of object, for any one condition of sight distance and algebraic difference in grades, reveal that the required length of vertical curve diminishes very rapidly as the height of object is increased from 0 to about 6 inches; for greater object heights, the reduction in length of vertical curve is progressively less significant. Substantial economy in construction (as reflected in the depth and volume of excavation due to shortening of vertical curve) is effected by using a 6-inch object instead of the desirable zero value, yet the ability to see or appraise a hazardous situation is not materially altered. A height of 6 inches is assumed for measuring stopping sight distance on crest profiles.

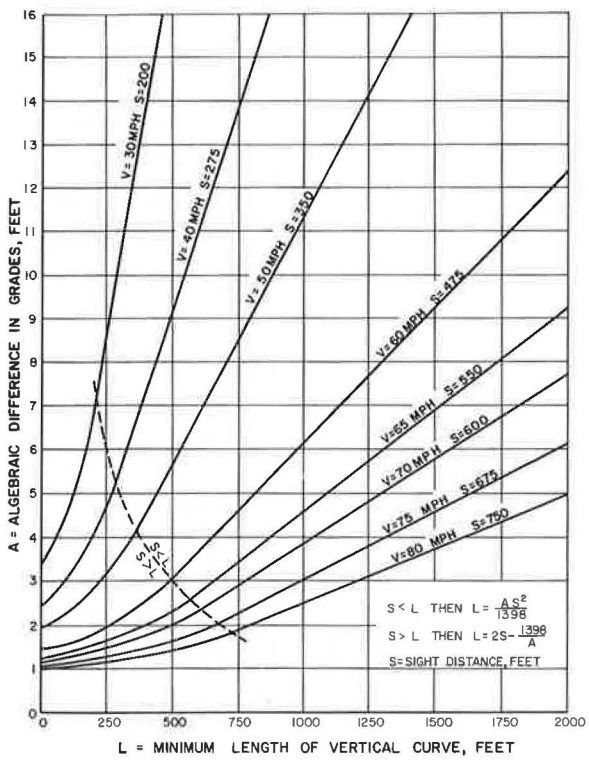


Figure 5. Relationship between stopping sight distance and length of crest vertical curve for eye height of 3.75 ft and object height of 0.5 ft.

These criteria for height of eye and height of object represent a departure from the 1954 AASHO Policy. For the 1954 Policy, the height of eye was assumed to be 4.5 ft. The same analysis as described earlier was used in the 1954 Policy to conclude that a 4-in. object height should be used in design. If 3.75 ft for the height of eye and 0.5 ft for height of object are used, the general equations for the length of vertical curve may be modified as follows:

$$L = 2S - \frac{1,398}{A} \quad \text{for } S > L$$

$$L = \frac{AS^2}{1,398} \quad \text{for } S < L$$

Figure 5 shows the relationship between length of vertical curve and sight distance for various algebraic differences in grade.

The height of object in determining stopping sight distance on horizontal highway curves is not as significant as that on vertical curves. Where the lateral sight obstruction is vertical, all heights of object may be seen at the same distance. Where the obstruction is an inclined cut slope, sight

distance is affected somewhat by the height of object, but the effect is not large. For consistency, the AASHO Policy uses the same height criteria on both horizontal and vertical curves, i.e., 3.75-ft eye height and 6-in. object height.

EVALUATION OF STOPPING SIGHT DISTANCE DESIGN STANDARDS

This discussion includes the evaluation of the validity of the following criteria employed in the AASHO stopping sight distance standards: perception and brake reaction times, braking distance equation, design values for the friction factor, assumed speeds for design, driver eye height, and object height.

Perception and Brake Reaction Time

The time interval of the stopping process, commonly called the perception-reaction time, is a very complex phenomenon. It is highly variable, dependent on the driver's psychological and physiological characteristics as well as the condition to be perceived. This may explain the lack of research to measure driver perception-reaction values in actual highway driving situations.

There are, however, conceptual explanations of the perception-reaction phenomenon. For example, Matson, Smith, and Hurd (12) describe the phenomenon as being composed of 4 elements: perception, intellection, emotion, and volition. Perception time is described as the time interval between the visibility of an object and the recognition of the object through visual sensation by the driver. The intellection time is that interval required for comparing, regrouping, and registering new sensations. Emotion is described as a time modifier of perception and intellection, dependent on the psychological makeup of the driver. Volition time is that interval necessary to exercise the decision to act. Another conceptual explanation of the perception-reaction process is offered by Baker and Stebbins (13) and is shown in Figure 6.

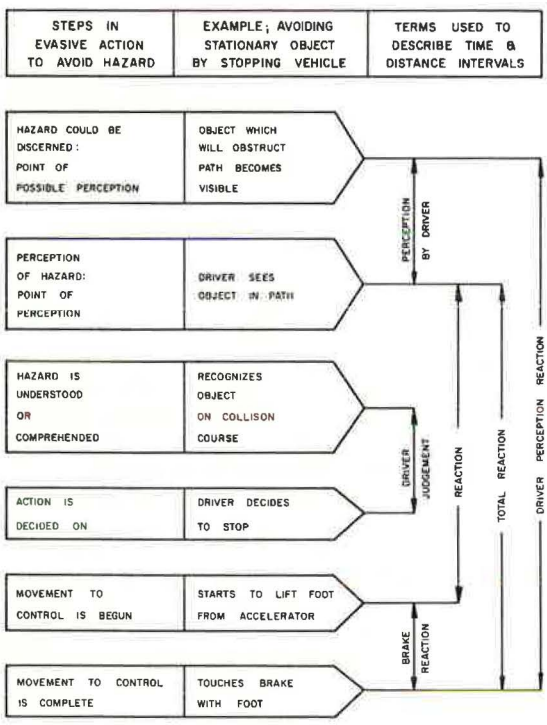


Figure 6. The perception-reaction process.

Matson, Smith, and Hurd (12) described many variables that affect perception and reaction time, including fatigue, physical disabilities, alcohol, drugs, climatic conditions, light conditions, and driver traits. Mullins (3) states that eye blinking occurs in intervals of 2.8 to 3.8 sec with a duration of 0.3 sec or more. He concludes that vision is unreliable for a short time before and after a blink and the modified blackout period caused by blinking may vary from 14 to 20 percent of all seeing time.

The most important element in the perception-reaction pheonomenon is perhaps the perception of form. Perception of form depends mainly on a sharp difference of brightness between an object and its background. Color difference is not as perceptible as brightness. Surfaces that differ in hue, but not in brightness, may be difficult to distinguish. In addition, highway design criteria assume that, as an object first comes into view on a crest vertical curve, the driver will perceive it and take appropriate action. Surely the driver has to see more than the first fraction of an object before he can perceive it. At 70 mph on a vertical

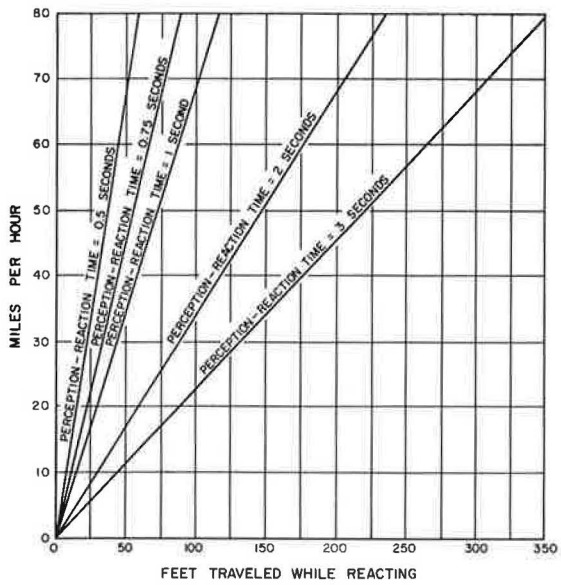


Figure 7. Distance traveled during various perception-reaction times for various speeds.

curve with a 600-ft stopping sight distance (i.e., the driver sees the top of a 6-in. object at 600 ft), the driver will not see the whole object until he has traveled 225 ft further.

The laboratory tests (4, 5) previously discussed indicated driver perception-reaction times in the range of 0.4 to 1.7 sec. As stated in the AASHO Policy, however, these times may be significantly greater for actual highway driving conditions. The AASHO Policy value of 2.5 sec, however, was hypothesized based on these studies. The perception-reaction time is highly variable, however, and, under critical conditions, could be higher than the 2.5-sec value.

Figure 7 shows how the distance traveled during perception-reaction time varies with speed. The required stopping sight distance would not be significantly changed for the lower speeds by increasing the required perception-reaction time by 1 sec. For the higher speeds, however, the distance is significantly increased for a 1-sec increase. Because of the trend of increased speeds on highways and because of degradation in visual acuity with higher speeds, the 2.5-sec perception time is questionably low for use in designing stopping sight distance for high-speed roadways.

Friction Factor Design Values

The AASHO Policy considered several studies (5, 8, 9) in determining the design values for friction factors (Fig. 2 and Table 1). The friction factor versus speed relationship employed is representative of wet pavements measured in the referenced studies. Because wet values are considerably lower than dry values, the wet values are a rational basis for design. The question remaining, however, is, Does this friction factor versus speed relationship represent a typical critical condition? Figure 8 shows a percentile distribution of skid numbers (skid numbers are considered equivalent to 100 f) at various speeds measured on 500 pavements (when wet) randomly dispersed throughout one state (14). These measurements were made in 1964 by using a modified ASTM skid trailer with standard ASTM test tires. Figure 2 shows that the curve that AASHO considers typical represents about the 35th percentile pavement in Figure 8. In other words, 35 percent of the pavements have friction factors lower than the design values. As such, the AASHO values are somewhat high. A more appropriate measure might be the 15th percentile pavement. Table 3 gives the friction factor versus speed relationship for 15th percentile pavement.

Stopping Distance Equation

If the relationship (shown in Fig. 2) of friction factor versus speed were indeed a typical critical curve, there would be no need for a verification of the validity of the stopping distance equation. The reason is that the AASHO Policy employed the equation to compute friction factors from the referenced stopping distance measurements and used these friction factors, with a safety margin applied, to recompute the stopping distance for design. In reality, this simply amounted to applying a safety factor to the original measured stopping distances.

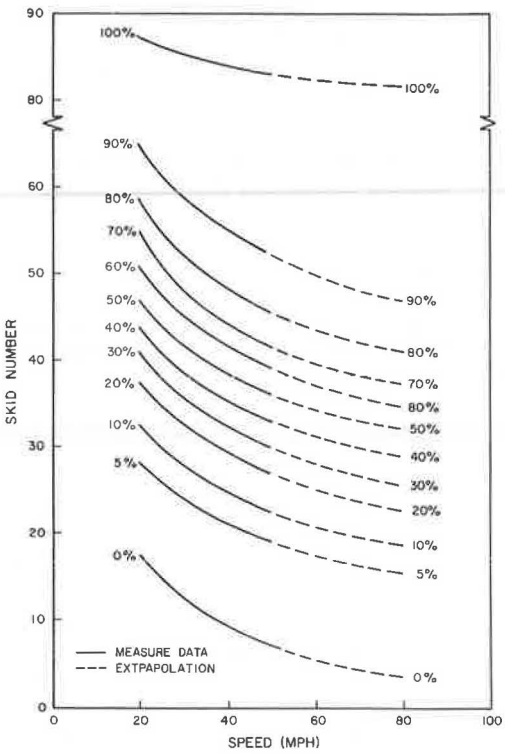


Figure 8. Percentile distribution of the skid number versus speed relationship for 500 pavements in one state.

TABLE 3

FRICTION FACTOR VALUES FOR THE 15TH PERCENTILE PAVEMENT IN ONE STATE

Design Speed	Friction Factor	Design Speed	Friction Factor
30	0.30	60	0.23
40	0.26	70	0.22
50	0.24	80	0.21

Because of the apparent need to consider lower friction factors (discussed previously), it is necessary to validate the equation for use of skid trailer measurements to compute stopping distance capability. Fortunately, tire quality test measurements (15) conducted by the Texas Transportation Institute in 1968 were available for analysis. The results of these measurements are given in the Appendix to this report.

The analysis considered 3,900 measurements of stopping distances of several tire brands and tire types on 5 test pavements of varying friction factors. Friction factors were measured (using the Texas skid trailer) on all 5 pavements periodically during the 6 months of testing. The analysis compared measured stopping distances with the stopping distances computed by using the appropriate friction factor in the standard equation. Although there was considerable variation between measured and computed distances, the analysis illustrated that only 10 percent of the trials involved stopping distances greater than those predicted by the equation.

In many of the trials, the vehicle stopped much shorter than predicted. This may be attributed to many variables such as experimental error, tire temperature, tire type, or pavement macrotexture. A considerable portion of the variation was attributed to measurements made on the pavement with the lowest friction factor (pad 4, approximate f-value of 0.20). On this pavement, the stopping distances in many trials were much shorter than those predicted by the equation. On the other 4 test pavements (f-values ranging between 0.44 and 0.64), the variation was considerably less than that on pad 4. The explanation for this phenomenon is apparently related to the accuracy of skid trailer measurements on lower coefficient pavements.

In summary, it appears that employing skid trailer values in the stopping distance equation yields reasonably conservative friction factor values for design purposes. The friction factors thus obtained should provide values of stopping distances that will be adequate for design of almost all conditions.

Assumed Speed for Conditions

The AASHO Policy states that it is not realistic to assume that travel will occur at full design speed when conditions are wet. Therefore, the Policy subjectively determined that the critical speeds for wet conditions should approximate 80 to 93 percent of design speed, as represented by average running speed for low-volume, or dry, conditions as shown in Figure 3. The AASHO Policy stated that this curve was developed from field data that related average spot speed to the design speed on horizontal curves. How ironic that the speeds taken from the curve in Figure 3 were used for

stopping sight distance design but not for horizontal curve design (for which full design speed is used for calculations).

It may be true that a critical speed (such as the 85th percentile) for wet pavements on horizontal curves is approximated by the average speed for dry conditions. Stopping sight distance, however, is also a design consideration on tangent alignment. It is reasonable to assume that low-volume average and 85th percentile speeds will be higher on tangent sections than on horizontal curves, especially for the lower design speeds. For free-flowing conditions the horizontal curvature is actually the only feature that limits speed (with the exception of very steep grades). No matter what the overall design speed of a highway may be, the operating speed of long level tangent sections on that highway is not limited by the geometry.

There are many variables that affect the spot-speed distribution of a highway section, including traffic volumes, percentage of commercial vehicles, contiguous design speed, type of facility, amount of roadside development, weather conditions, wet pavements, and posted speed limits (16). Many studies have been conducted to relate traffic speeds to posted speed limits, but no references are available that have measured the relationship between critical wet weather speeds and design speed.

The Texas Highway Department (17) collected wet and dry weather speed data at 16 sites in 1968. Various speed distribution parameters taken from these measurements are given in Table 4. The speed stations employed in this study were essentially level-tangent sections to limit the variation due to geometric design. Also, measurements were made of free-flowing vehicles only to eliminate variation due to traffic friction. In most cases, the overall design speeds of the highways were at or below the posted speed. From the data given in Table 4 and the description of site and measurement conditions, the following observations have been made pertaining to wet weather speeds:

1. There appears to be a leveling off of the higher speeds as posted speed increases. This is illustrated by the 85th percentile speed variation for 50, 60, and 70 mph posted speeds.
2. There is more variation in the distribution parameters for wet weather conditions than for dry weather conditions. This may be due to variations in rainfall intensity.
3. If it may be assumed that posted speeds approximate design speed, then it appears that the 85th percentile wet weather speed closely approximates the design speed for lower design speed highways (less than 50 mph).
4. The wet weather 85th percentile speed averages about 3 mph higher than the dry weather average speed. This variation has an indicated regression that would vary from 0 mph at lower speeds to 5 mph at the higher speeds.

TABLE 4

SPOT-SPEED DISTRIBUTION PARAMETERS FOR WET AND DRY WEATHER CONDITIONS

Site	Site Conditions		Wet Weather Speed Percentile			Dry Weather Speed Percentile			
	No. of Lanes	Posted Speed	50th	85th	90th	Average	50th	85th	90th
1	4	50	45	51	53	51	50	58	60
2	4	60	47	55	57	52	51	58	60
3	4	60	50	58	60	63	61	70	72
4	4	60	55	62	64	59	58	63	66
5	4	60	56	63	65	60	59	65	67
6	4	60	58	68	70	60	60	70	72
7	4	60	60	67	69	66	65	71	74
8	4	60	61	68	69	64	64	70	71
9	4	60	64	71	73	67	66	73	75
10	4	70	52	59	62	61	61	68	70
11	4	70	52	62	64	57	56	63	65
12	4	70	54	61	63	58	57	67	68
13	2	70	54	64	65	59	59	69	70
14	4	70	56	64	65	62	61	70	72
15	4	70	55	64	65	60	60	70	71
16	2	70	56	64	66	61	61	70	71

TABLE 5

A DERIVATION OF CRITICAL WET SPEEDS FOR DESIGN

Design Speed (mph)	AASHO Average Dry Speeds on Curves (mph)	Assumed Difference Between Average Dry Speeds on Tangent and Curves (mph)	Derived Average Dry Speeds on Tangents (mph)	Assumed Difference Between Average Dry Speed and 85th Percentile Wet Speeds on Tangents (mph)	Derived 85th Percentile Wet Speeds (mph)
30	28	6	34	0	34
40	36	6	42	0	42
50	44	6	50	0	50
60	52	5	57	2	59
65	55	4	59	3	62
70	58	3	61	3	64
75	61	2	63	4	67
80	64	1	65	5	70

The AASHO Policy's "assumed speeds for condition" have no objective basis. The question then is, What speeds should be used for design? One argument holds that the design speed should be used as the critical design speed for stopping sight distance, just as it is used for horizontal curve design. This could be justified on the basis that, at some time, a highway might be expected to have a posted speed limit either at or above the design speed. In addition, it might be expected that some drivers will exceed the posted limit, regardless of weather conditions.

Another basis for critical speeds to use in stopping sight distance design could be derived from the low volume curve in Figure 3 and from the Texas speed data. This would require that assumptions be made regarding the relationship between average dry speeds on tangents and average dry speeds on horizontal curves. These assumed differences are given in Table 5, along with the AASHO low-volume average dry speeds on horizontal curves, the derived low-volume average speeds on tangents, the differences between average dry speeds and 85th percentile wet speeds on tangents (discussed previously), and the derived critical speeds assumed for design. With this method, the critical speeds are somewhat higher than those in the AASHO policy.

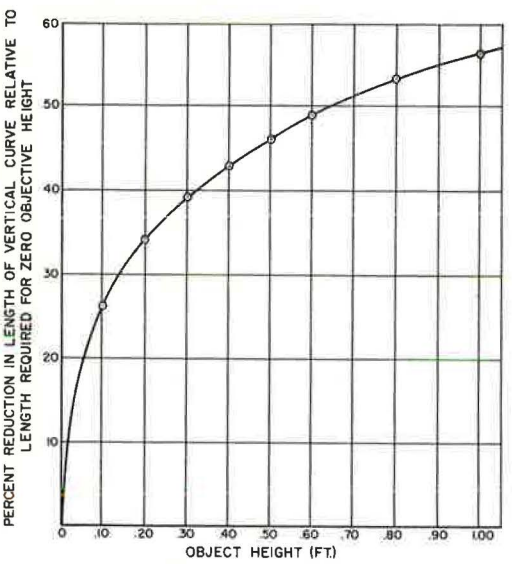


Figure 9. Relationship between object height and required length of crest vertical curve.

Driver Eye Height

It appears that the driver's eye height has not been significantly lowered since the 1960 automobile models. Therefore, the 3.75-ft eye height may be a reasonably valid criterion for the design of stopping sight distance; however, the percentile distributions shown in Figure 4 are for models and are not distributions experienced on the highway. It is possible that a considerable percentage of driver eye heights on the highway are lower than 3.75 ft because of the introduction and high-volume sales of automobiles such as the Ford Mustang and the Chevrolet Camaro.

Object Height

The AASHO Policy considers that a zero object height would provide for the safest sight distance design. The 6-in. object height, however, was selected because it supposedly represented a point of diminishing returns in terms of the cost of excavation, considering the relationship between the object height and the length of

TABLE 6

SAFE STOPPING SIGHT DISTANCES FOR HEAD-ON COLLISION CRITERION

Design Speed (mph)	Assumed Speed for Condition (mph)	Perception-Reaction		Friction Factor (f)	Braking Distance on Level (ft)	Stopping Sight Distance		
		Time (sec)	Distance (ft)			1 Vehicle (ft)	2 Vehicle (ft)	Rounded for Design (ft)
30	32	2.5	117	0.29	118	235	470	470
40	40	2.5	147	0.26	205	352	704	700
50	48	2.5	176	0.24	320	496	992	1,000
60	57	2.5	209	0.23	471	680	1,260	1,260
65	60	2.5	220	0.23	522	742	1,484	1,480
70	62	2.5	228	0.23	557	785	1,570	1,570
75	65	2.5	239	0.22	640	879	1,758	1,760
80	68	2.5	250	0.22	700	950	1,900	1,900

vertical curve required to provide stopping sight distance for various object heights. This relationship between object height and length of vertical curve is shown in Figure 9 (1). The point of diminishing returns appears to be more nearly in the range between 0.1 and 0.3 ft.

This basis for designing vertical curvature may provide safety for most operational conditions; however, whether it is entirely adequate for providing relatively high overall safety of operation is questionable. There appear to be many operational situations that would require a zero object height for safety, such as either a horizontal curve or an intersection hidden by a crest vertical curve. Therefore, the present criterion for object height bears no relation to many of the operational requirements for safe stopping sight distance.

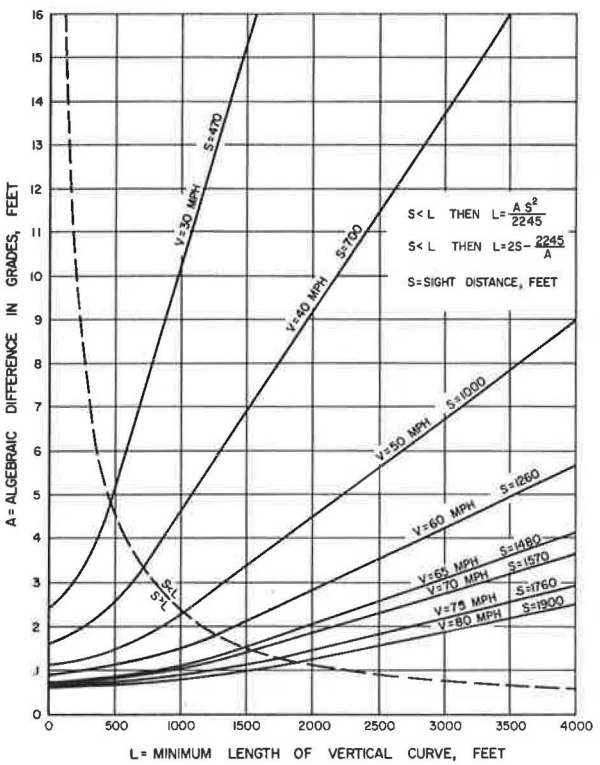


Figure 10. Relationship between stopping sight distance and length of vertical curve for head-on collision criterion.

Head-on Collision Criterion for Stopping Sight Distance

At this point, an entirely new philosophy of stopping sight distance design will be offered. On 2-lane highways, a vehicle must travel in the opposing lane to pass a slower moving vehicle. Legally, this is only possible where adequate passing sight distance is available and no restrictions are placed on passing. Traffic engineers and traffic law enforcement officers, however, well know that no-passing zones are often violated. Considering those drivers who violate no-passing zones and also drivers who wander into the opposing lane because of drowsiness or intoxication (2 characteristics that are all too common, especially at night), we might appropriately design for nighttime stopping sight distance to opposing headlights, allowing for the closing rate of the 2 vehicles. It is true that a driver can see the beams of opposing headlights well before he can see the headlights themselves. He cannot always perceive, however, that the opposing vehicle is in his lane until

TABLE 7

SAFE STOPPING SIGHT DISTANCES FOR STATIONARY OBJECT CRITERIA THAT EMPLOY DESIGN SPEEDS FOR WET CRITICAL SPEEDS

Design Speed (mph)	Assumed Speed for Condition (mph)	Perception-Reaction		Friction Factor (f)	Braking Distance on Level (ft)	Stopping Sight Distance	
		Time (sec)	Distance (ft)			Computed (ft)	Rounded for Design (ft)
30	30	2.5	110	0.30	100	210	210
40	40	2.5	147	0.26	205	352	350
50	50	2.5	184	0.24	347	531	530
60	60	2.5	221	0.23	522	743	750
65	65	3.0	287	0.22	644	931	930
70	70	3.0	309	0.22	742	1,051	1,050
75	75	3.5	386	0.21	893	1,279	1,280
80	80	3.5	412	0.21	1,016	1,428	1,430

TABLE 8

SAFE STOPPING SIGHT DISTANCES FOR STATIONARY OBJECT CRITERIA THAT EMPLOY DESIGN SPEEDS FOR WET CRITICAL SPEEDS

Design Speed (mph)	Assumed Speed for Conditions (mph)	Perception-Reaction		Friction Factor (f)	Braking Distance on Level (ft)	Stopping Sight Distance	
		Time (sec)	Distance (ft)			Computed (ft)	Rounded for Design (ft)
30	34	2.5	125	0.28	138	263	260
40	42	2.5	154	0.26	226	380	380
50	50	2.5	184	0.24	347	531	530
60	59	2.5	217	0.23	504	721	720
65	62	3.0	273	0.23	557	830	830
70	64	3.0	282	0.23	593	875	880
75	67	3.5	345	0.22	680	1,025	1,030
80	70	3.5	360	0.22	742	1,102	1,100

the headlights are visible. The AASHO Policy states that headlight beam height is approximately 2 ft from the ground level.

For minimum safety requirements, the safe stopping sight distances for the head-on situation should be doubled. Although seeming to be more than enough, this is probably conservative because it assumes that the opposing vehicle will stop, and it may not if the driver is either intoxicated or asleep. In designing for the head-on situation, one is again faced with the problem of establishing critical speeds for design. In this case, the critical speed should be, for example, the 85th percentile night wet weather speed. The 1968 speed survey (18) by the Texas Highway Department shows that the nighttime 85th percentile speed is about 2 mph lower than the daytime 85th percentile speed. If the assumptions in Table 5 are accepted, then the 85th percentile night wet weather speed might be derived as 2 mph lower than the 85th percentile day wet weather speeds.

The 2.5-sec perception-reaction time (assumed by AASHO) is probably low for the higher travel speeds, but that evaluation was based on a stationary object without its own source of illumination. For stopping sight distance design to an opposing vehicle, the 2.5-sec perception-reaction time is probably adequate. Table 6 gives the proposed safe stopping sight distances for the head-on collision criterion employing the assumptions discussed in this subsection. The design friction factors represent the 15th percentile pavement as previously discussed (Table 3). Figure 10 shows the relationship between stopping sight distance and length of vertical curve for the head-on collision criterion.

Stationary Object Collision Criterion for Safe Stopping Sight Distance

For multilane, divided, and possibly undivided highways, there is no apparent need for the head-on collision criterion. There are, no doubt, other operational conditions

that have respective stopping sight distance requirements. Until these operational requirements can be defined, the stationary object criterion is all that is available.

Table 7 gives stopping sight distances based on the assumption that the critical speed is equivalent to the design speed. Table 8 gives stopping sight distances based on the critical speeds derived in Table 5. In both Tables 7 and 8, the 15th percentile pavement given in Table 3 is employed for friction factor values. In addition, the perception-reaction times for the higher speeds have been adjusted upward, in light of the earlier discussion in this section.

CONCLUSIONS

1. The commonly used criterion of a 2.5-sec perception-reaction time for the braking maneuver was based on a subjective extrapolation from laboratory studies. This evaluation indicates that the perception-reaction time is highly variable and, under critical conditions, could be higher than 2.5 sec. Because of the trend toward higher speeds and the concomitant degradation of a driver's visual acuity with higher speed, the 2.5-sec criterion is questionable for use in designing stopping distance for high-speed roadways.
2. Based on skid trailer measurement of 500 pavements randomly dispersed throughout one state, the AASHO design friction factor values do not represent a critical level of stopping capability. The AASHO values are representative of the 35th percentile pavement in the one state. In other words, 35 percent of these pavements could not provide adequate stopping distance to meet minimum stopping sight distance standards.
3. The use of skid trailer values (which are related to design speed) in the standard stopping distance equation will yield reasonably conservative stopping distance values for use in design. This evaluation was based on 3,900 stopping distance tests, which were related to computed stopping distances derived from test speed and a representative friction factor versus speed relationships for the test pavements.
4. The AASHO Policy's "assumed speed for conditions" has no objective basis. Because of lower friction factors when pavements are wet, the wet condition is the rational basis for design. The AASHO Policy states that it is not realistic to assume that travel will occur at full design speed when conditions are wet; however, the "assumed speed for conditions" is arrived at by employing average speeds for dry conditions on horizontal curves (of given design speed) as the critical wet speeds (related to design speed) for stopping sight distance design. Other bases for determining the "assumed speed for conditions" are presented.
5. A 3.75-ft driver's eye height is employed in the measurement of stopping sight distance design. This height is representative of the distribution of 1960-model automobiles. Although no data are available, this eye height is reasonably representative of current production automobiles. An eye height representative of vehicles on the roadway could be lower, however, because of the introduction and high-volume sales of automobiles such as the Ford Mustang, the Chevrolet Camaro, and the Volkswagen.
6. Theoretically, a zero object height would provide the safest sight distance design. The 6-in. object height used for the measurement of stopping sight distance design was supposedly selected on the basis of diminishing returns, in terms of the cost of excavation for crest vertical curves. Because excavation is no longer the major cost of highway construction, a more appropriate object height would be in the range of 0.1 to 0.3 ft.
7. The present criterion for object height bears no relation to many of the operational requirements for safe stopping sight distance. There appear to be many operational conditions that require a zero object height for maximum safety, such as a horizontal curve or an intersection hidden by a crest vertical curve.
8. There is a need to design nighttime stopping sight distance for opposing vehicle headlights on 2-lane highways. It is not uncommon for vehicles to be in the opposing lane even if there are legal restrictions. The opposing driver may be asleep, intoxicated, or otherwise openly violating the restriction.

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Appendix

TIRE TEST RESULTS

The 1966 National Traffic and Motor Safety Vehicle Act provided for the development of a uniform-quality grading system for pneumatic passenger-vehicle tires. In order to develop this system, the National Bureau of Standards conducted tests on tires currently in production to provide the necessary data base. Under contract to the National Bureau of Standards, the Texas Transportation Institute undertook the testing of 95 sets of tires during the period from March through November 1968 (15). The various sets of tires included in this program are given in Table 9. Each set of tires was tested to provide data on tractional characteristics when stopping with locked wheels and to determine loss of traction when driving through curves.

The pavements used in this test program were specially designed to achieve predetermined coefficients of friction. They included 4 different asphalt pavements and 1 portland cement concrete pavement. Each stopping pad was 24 ft wide and 600 ft long, having a cross slope of 2 in. in 24 ft.

Test Vehicle

The automobile used in this test program was a 1968 4-door Bel Air Chevrolet. Modification was made to the suspension system, including a change to heavy-duty coil

TABLE 9

TIRE SETS TESTED

Tire	New	Mileage Worn	Random Rerun (new and mileage worn)
Bias ply	20	13	5
Radial	8	8	4
Wide oval	12	0	2
Snow	11	0	2
Police	2	0	1
SAE	4	0	2
Wide slicks	1	0	0
Total	58	21	16

springs and heavy-duty shock absorbers. Prior to each day of testing, the vehicle height was determined by measuring the height of marks placed on the bumper at each corner of the car. This procedure was established to determine if deterioration was occurring in the suspension system. Air pressure for the automobile tires tested was 24 psi cold.

The tire-test vehicle was equipped to indicate and record the following information: distance traveled as a function of time, velocity of the vehicle as a function of time, rear-wheel lock-up point, and lateral forces (transverse accelerations). Distance and velocity data were obtained from a track-test fifth-wheel assembly attached to the rear bumper. Data were recorded on a Honeywell Visicorder. The ac power required for the Visicorder was supplied by a gasoline engine generator mounted in the trunk.

Test Surfaces

The location of the Texas A&M Research Annex on property that had previously been a jet trainer airfield permitted a wide choice in the specific location of the various test pavements. The study called for the design and construction of 4 different surface textures produced from selected aggregate and a single grade and type of asphalt cement. The program also required a fifth surface that consisted of selected portions of the existing portland cement concrete runways. These different surfaces were expected to have particular coefficients of friction that would remain constant for the duration of the study.

The preparation of the existing portland cement concrete pad consisted of a thorough cleaning. The other surfaces were to be designed and constructed to provide a range of friction coefficients between 0.20 to 0.60. Pavements were produced that covered a range of 0.18 to 0.64 (as measured with a skid at 40 mph) at the beginning of testing.

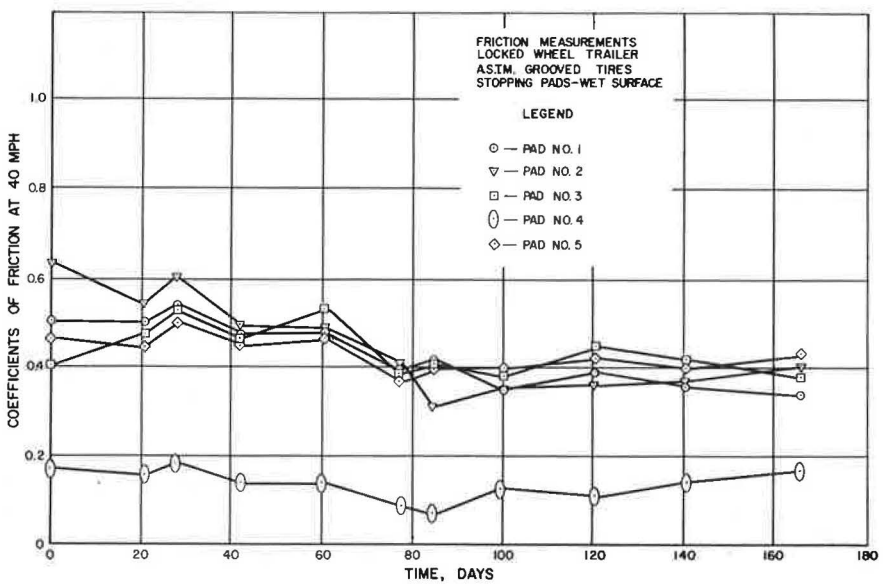

Figure 11. Variation of test pad coefficients over period of testing.

TABLE 10

LOCK-UP VEHICLE SPEEDS FOR EACH TEST PAD

Pad	Lock-Up Speeds (mph)			Pad	Lock-Up Speeds (mph)		
1	30	40	50	4	15	25	35
2	35	45	55	5	30	40	50
3	30	40	50				

The spread of these coefficients was reduced during the course of the project to a range of 0.18 to 0.44. The friction coefficients over the period of testing are shown in Figure 11 for each test pavement.

Skid Trailer Measurements

The friction values shown in Figure 11 were obtained with the Texas Highway Department skid trailer run with standard ASTM grooved test tires. The source of water for wetting the pavements was a 4,000-gal water truck complete with spray bars and a controlled pumping system capable of producing a uniform flow and distribution of water. On each pad, 1 pass of the watering truck preceded 2 passes with the skid trailer (one in each direction). The skid trailer's self-watering system was not used.

Friction determinations were made at 20, 30, and 40 mph. For each speed, 6 locked-wheel skids were made on each pad (3 in each direction). The points plotted in Figure 11 represent averages of these 6 measurements.

Stopping Distance Tests

The test vehicle was subjected to 4-wheel lock-up 4 times for each of the speeds given in Table 10. Two test trials were conducted in each direction for each pad, tire set, and speed. The speeds given in Table 10 were determined at the beginning of the test program, so the driver would not be subjected to extremely unsafe conditions.

On pads 1, 3, 4 and 5, 1 pass of the watering truck preceded each 2 passes of the test vehicle (once in each direction). On pad 2, the same procedure was followed, except that, prior to the 55-mph test run, 1 pass of the watering truck preceded each trial run. This was necessary because of the nature of the pavement and time consumed by the driver in accelerating to the 55-mph speed.

Approximately 3,900 total stopping distance measurements were made on the following tire set types: new bias-ply, worn bias-ply, new radial, worn radial, and new wide oval.

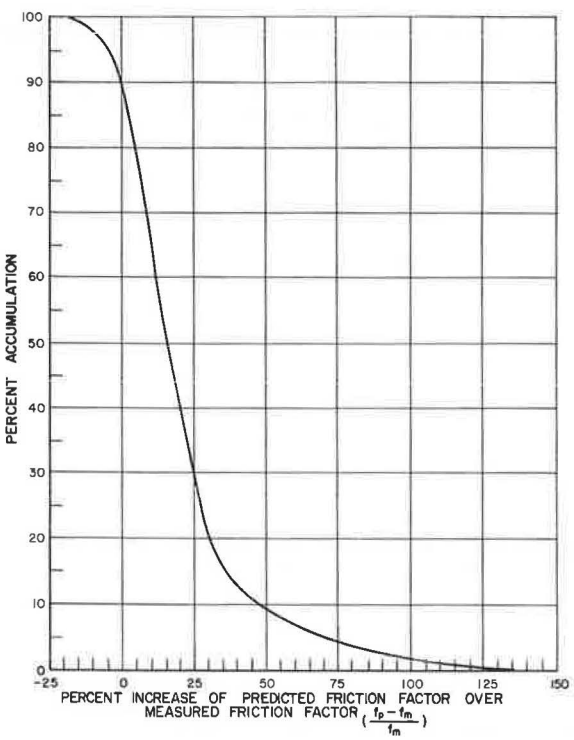

Figure 12. Percentile distribution of percentage deviation between measured and computed friction factors for all tests.

Test Results

For each stopping distance test run, it was possible to associate a computed friction factor with the test speed, based on the skid trailer measurements made at 20, 30, and 40 mph. The computed friction value and the test speed were used to compute the predicted stopping distance by using the standard equation $d = V^2/30f$. The prediction reliability of the equation could then be analyzed by comparing the predicted values with the measured values.

Figures 12 and 13 show percentile distributions of test runs related to

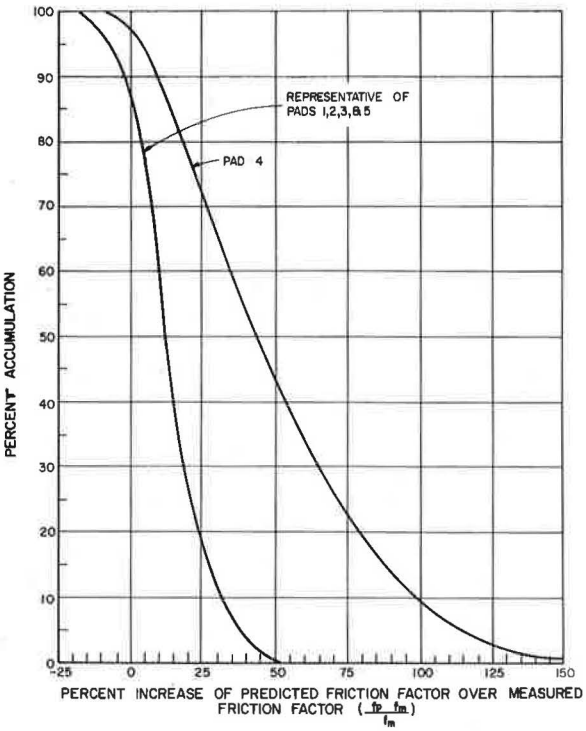

Figure 13. Percentile distribution of percentage deviation between measured and computed friction factors by test pad.

the percentage difference between the measured friction factor and the computed friction factor, (fc - fm)/fm. In 90 percent of the 3,900 test trials, the vehicle stopped in a shorter distance than that predicted by the standard stopping distance equation (Fig. 12). Figure 13 shows that, for the lowest coefficient pavement (pad 4), the vehicle stopped considerably shorter than predicted. No explanation can be offered for this phenomenon.

Additional analyses of these data indicate that (a) the equation is somewhat more reliable for lower stopping speeds; (b) new radial and new wide oval tires have a somewhat better stopping capability than new bias-ply tires; and (c) new tires have a somewhat better stopping capability than worn tires (10,000 high-speed miles of wear).

Discussion

D. W. LOUTZENHEISER, Highway Standards and Design Division, Bureau of Public Roads, Federal Highway Administration, U. S. Department of Transportation—Glennon's evaluation of sight distance design criteria is very timely because the AASHO Committee on Planning and Design Policies, which prepared the document examined, is now engaged in revising the AASHO policy on stopping sight distances.

In the AASHO concept for highway design, the minimum sight distance available on a highway is made sufficiently long to enable a vehicle traveling at or near the likely top speed to stop before reaching an object in its path. Although 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.

Minimum stopping sight distance for a stated speed is derived as the sum of 2 distances. One is the distance traversed by a vehicle from the instant the driver sights

an object for which a stop is necessary until he decides to act. The other is the distance required to stop the vehicle after the brake application begins. The design control is a minimum value for each design speed, to be used for that speed on any highways and for all conditions. Derivation of these values requires determination of 3 factors over the range of speeds used in design, namely, perception and reaction time, coefficient of friction, and vehicle speed. The first 2 factors lend themselves to scientific measurement under controlled laboratory conditions, but evaluation of such measurement for general highway conditions is difficult because of variations in physical properties of tires and roadway surfacing materials, physiological differences among drivers, weather, and the highway geometric details. Judgment, therefore, plays a major role in the determination of representative values for needed length of sight distance to ensure reasonably safe operation and at the same time be economically attainable.

Glennon lists 8 conclusions from his evaluation. Each is worthy of comment.

1. He questions the adequacy of 2.5 sec for perception-reaction. Admittedly, there is little factual support for 2.5 sec, and this value as well as any other that might be proposed is subject to conjecture. Results of actual investigations would be welcomed. To simply question the 2.5 sec offers little guidance as to a correct value. There are no plans to alter the 2.5-sec value in the revision of the AASHO sight distance standards.
2. He points out that the friction factor used in the stopping distance equation is higher than the skid numbers for 35 percent of pavements tested in one state. He suggests that a more appropriate measure might be the 15th percentile pavement. This is very useful information, and his suggestion seems to be a sound one. Each state, and perhaps each highway district within a state, could utilize the results of friction tests to determine sight distance requirements.

Two additional comments seem appropriate. First, the friction factor in the equation is the average value over the entire length of stop. Because friction increases as speed decreases, the average friction is considerably higher than the skid number measured at uniform speed. Thus, a friction factor (average) of 0.33 as used in the AASHO Policy for a stop from 40 mph may not be out of line with a factor of 0.26 measured at a uniform speed of 40. There is need to develop better correlation between actual stopping distance and calculated stopping distance based on friction factors measured with skid trailers. The former involves variables such as brake fade and tire-pavement relations that are not measured by the uniform speed method (see also item 4).

The second comment is that a long sight distance cannot ensure that a slippery pavement is a safe pavement, although a long sight distance may help. It might be wiser to devote a larger share of the highway dollar to improving skid resistance than to providing long sight distances. Such antiskid improvements could be programmed on a selective basis, if desirable, so that pavements would be treated first where sight distances are near minimum.

3. He shows that the use of skid numbers obtained with skid trailers will yield reasonably conservative stopping distance values for use in design. In many of the trials the vehicle stopped much shorter than predicted. This reassuring bit of information is welcome news. The reason for the conservatively low stopping distances may be as suggested in the first comment for item 2.
4. He concludes that the AASHO Policy's "assumed speed for conditions" has no objective basis. Interestingly enough, the results given in Table 4 support the Policy's position that speeds are somewhat lower when pavements are wet than when dry. The attempt by the author to relate measured speeds to design speed on the assumption that posted speed limits are approximately equal to design speeds is debatable. The AASHO Committee is proposing that the vehicular speed to be used in the stopping sight distance equation should be the same as the design speed of the highway, even with wet pavements. These speeds are, of course, considerably higher than those utilized in the development of current AASHO standards and somewhat higher than those proposed by Glennon.
5. He observes that the driver's eye height could be lower now than when data were compiled for the AASHO Policy. This is true; however, the eye height of 3.75 ft is

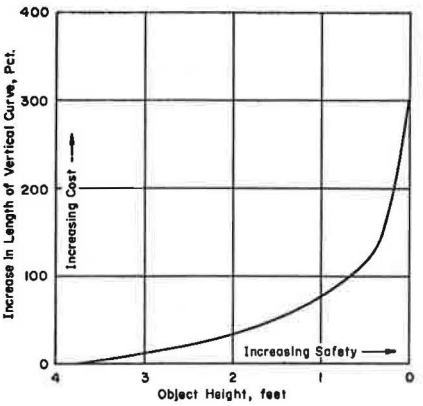

Figure 14. Relationship between object height and length of vertical curve needed to provide safe stopping distance (based on eye height of 3.75 ft and maximum object height of 3.75 ft).

lower than that found for the Corvair, Falcon, Valiant, and other compact cars that were in production at the time the standards were last revised. It is well below that for the Volkswagen and is believed to be on the conservative side for all but a very few foreign-made sports cars.

6 and 7. He proposes an object height in the range of 0.1 to 0.3 ft. Of all the variables that enter into the determination of minimum lengths of vertical curves, this one is least adaptable to scientific derivation. The value of 0.5 ft is admittedly a compromise that is vulnerable to challenge. Glennon arrived at his suggested values by what he refers to as the point of diminishing returns. However, his application of the economic law of diminishing returns is somewhat at variance with normal procedures. Usually, the point of diminishing returns is said to be reached when a relatively large expenditure is needed to purchase a relatively small benefit. In this case the length of vertical curve is a measure of expenditure and the benefit, which is the increase in safety, is measured by a lowering of the object height. Viewed in this sense, the point of diminishing returns appears to be reached at an object height of about 0.5 ft (Fig. 14). It is actually somewhat higher than this, however, because earthwork quantities increase almost exponentially with increasing lengths of vertical curves. Consequently, costs rise more rapidly than do the lengths of curves.

Glennon also suggests that a zero object height should be applied at horizontal curves. The AASHO sight distance design bases is that of a "single" minimum value for each design speed, applicable to govern design of both crest vertical curves and horizontal curves. In the original derivations (about 1940) it was concluded both practical and desirable to avoid a double sight distance standard. Instead the criteria for height of driver's eye and height of object were established as logical values that compromise the 2 curve conditions. Thus the design sight distance value derivation involves a "package" of interwoven factors—the 3 factors for stopping distance and the 2 factors for height—all of which must be jointly considered in any change. The intricacies of the interrelations could be more fully explained at some length, but further review is not made here.

8. He develops an entirely new philosophy for needed stopping distance for 2-lane roads. This is both refreshing and challenging. It is refreshing for the reason that the need for a new philosophy for both 2-lane and multilane highways is becoming increasingly apparent, and it is good that Glennon is willing to take the first step. Perhaps instead of stopping distance, the need is for maneuver distance or maneuver time or room. Possibly the most likely object to be dealt with then is another automobile. Even on 2-lane highways, a stopped vehicle in the travel lane may be a more likely hazard than an oncoming one. Regardless of what type of object is chosen, there is need to know how large the target must be before the driver can detect that a stop (or other maneuver) is called for. If, for example, a car is the critical object, must the driver see all above the headlights, even in daylight, before he can detect that evasive action is necessary? It seems reasonable that he might.

Glennon's philosophy is challenging in that some of his assumptions are controversial, and necessarily so. For example, he assumes that the minimum sight distance, driver eye to headlight height, should be double the stopping distance. Presumably this is based on the theory that both the "safe" driver and the errant driver will detect that an emergency stop is necessary and that their bumpers will just touch as they come to a halt. If each is traveling 65 mph, they can be no closer than $\frac{1}{3}$ mile apart when they commence this action. In the glare of oncoming headlights, or on a slight horizontal

curve, the drivers are a long distance away to detect a hazardous condition calling for immediate action. It hardly seems logical to expect that the driver who is at fault will remain in the wrong lane until the 2 cars meet, and even less likely that he will attempt to stop in that lane. If, however, he should continue on the wrong side of the centerline at a speed of, say, 60 mph and if the safe driver attempts to stop from, say, 60 mph, he will still be traveling at 16 mph when the 2 vehicles meet.

I hope that this report will stimulate action to further explore Glennon's philosophy and to develop new ones supported by research findings. Information relating accidents to sight distances would be a great help in deciding on minimum sight distance standards, but such information is scarce. A recent report is, however, rather enlightening in this regard and is worthy of mention (19). Although this report does not show a definite cut-off point or point of diminishing returns beyond which accidents show a sharp increase with a further decrease in sight distance, it does make clear that the greater the minimum sight distance (up to 2,600 ft, which was the limiting value used in the study) the safer the highway becomes. This should impress upon everyone that above-minimum values should be used whenever feasible. It should also lend support to the higher sight distance standards under consideration by AASHO that, if adopted, will result in sight distances up to 40 percent longer than those now in use.

Reference

19. Cirillo, J. A., Dietz, S. K., and Beatty, R. L. Analysis and Modeling of Relationships Between Accidents and the Geometric and Traffic Characteristics of the Interstate System. U. S. Govt. Print. Office, Washington, D. C., Aug. 1969.